









## **SPECIAL NOTE.**

Part VI., relating to Stanchions, is compiled in accordance with the Amendment of London Building Acts, London County Council (General Powers) Act, 1909, Part IV., with respect to Buildings of Steel Skeleton Construction in London

Part II., also relating to Stanchions, complies with the London Building Acts, 1894 to 1908, and Provincial Building Requirements.

The remainder of the book is of general application.

**REDPATH, BROWN & CO., LIMITED.**

1915 EDITION  
(REPRINT OF  
• 2ND EDITION OF 10,000 COPIES).

# HANDBOOK OF STRUCTURAL STEELWORK.

**REDPATH, BROWN & CO., LIMITED**  
*(On the Admiralty, War Office and India Office Lists)*

HEAD OFFICE OF THE COMPANY :  
**2 ST. ANDREW SQUARE, EDINBURGH.**

WORKS AND STOCKYARDS :  
**LONDON, EDINBURGH,  
MANCHESTER, GLASGOW.**

*See overleaf for the full addresses of the various establishments  
of the Company.*

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**PREFACE, 1915 EDITION**

(Reprint of  
2nd Edition of 10,000 Copies).

Our object in issuing this Handbook of Structural Steelwork is to place before our clients, in as convenient a form as possible, all the data required for the design of Structural Steelwork.

We take this opportunity of emphasising our ability to give **immediate delivery.**

Our facilities in this respect are unique for several reasons:—

1. We have the largest and most varied stock of Structural Sections in the United Kingdom, amounting in the aggregate to over 20,000 tons.
2. In addition to our four Stock-yards, we have fully equipped Structural Works at each of our establishments.
3. On receipt of orders, our works obtain material from our stocks, and workmanship is commenced at once.
4. The usual vexatious delays, due to waiting for materials coming from rolls, are thus obviated.

A brief description of our stock is given on the following pages.

**RANGE OF STOCKS.**

The sections of Steel Joists, Channels, Angles, and Tees, which we stock, are those recommended by the British Engineering Standards Committee.

## REDPATH, BROWN & CO., LIMITED.

All sections are stocked in varying lengths up to the following limits :—

Steel Joists, - - - - -	up to 40' 0"
" Channels, - - - - -	up to 40' 0"
" Angles, - - - - -	up to 40' 0"
" Tees, - - - - -	up to 40' 0"
Rolled Edge Steel Flats and Plates, in all usual widths and thicknesses, - - - - -	up to 40' 0"
Round Bars for Solid Steel Columns, from 2½" upwards, - - - - -	up to 30' 0"
Bridge Rails, - - - - -	up to 40' 0"

We also stock Broad Flange Beams, particulars of which are given on page 11.

Complete Stock Lists may be had on application. Clients at home or abroad specifying these sections can depend on their requirements being supplied promptly.

Our usual Shipping Ports are :—

LONDON, LIVERPOOL, MANCHESTER, GLASGOW,  
MIDDLESBROUGH, AND LEITH.

## QUALITY OF STEEL AND TESTS.

The whole of our stock material is of uniform quality, and is supplied to us by the makers to the following tests, which are those accepted by the Admiralty and Lloyd's Inspectors, and are also in

## REDPATH, BROWN & CO., LIMITED.

accordance with the British Standard Specification for Structural Steel :—

Ultimate tensile strength not less than 28 tons nor more than 33 tons per square inch.

Elongation not less than 20 per cent. on a length of 8 inches.

All our stock material is bought on the understanding that it complies with these tests, and every precaution is taken by us to ensure this.

As much valuable time is lost when materials are ordered from the rolls, we strongly recommend our clients to specify "stock material" on all occasions. By so doing it is possible to take full advantage of our facilities for giving immediate delivery.

### ROLLING MARGIN.

In each case the weight per foot given in the tables is the minimum that can be rolled, and is subject to a rolling margin of  $2\frac{1}{2}$  per cent. over. This margin is claimed by the rolling mills, and should be allowed for in all calculations of weights.

### WEIGHT OF STEEL.

All weights per foot given in the tables are calculated on the basis that a cubic foot of steel weighs 489·6 lbs., i.e., a piece of steel 12" square by 1" thick weighs 40·8 lbs.

## **SPECIFIED LENGTHS.**

All sections, either from works or from stock, are cut to a margin of 1" over or under the specified lengths.

When sections are ordered to be cut exact, which means within  $\frac{1}{8}$ " of specified lengths, the usual extra is charged.

All orders are executed by us subject to the above conditions.

## **STOCK-YARDS AND WORKS.**

Our establishments at Edinburgh, London, Glasgow, and Manchester are each complete units, consisting of Commercial and Technical Offices, Stockyards, and Workshops. A general idea of the extent of these may be had from the photographs included in this Edition.

Our several works are fully equipped with the latest machinery for the rapid and accurate manufacture of all classes of structural Steelwork.

Efficient technical staffs, whose services are at the disposal of our clients, are maintained at all our works.

We are always willing to submit complete schemes or to give estimates from architects' or engineers' designs.

## SCOPE AND ARRANGEMENT OF THE BOOK.

Extended reference to the Contents is not necessary here, as an Index is provided, but the following general notes may be of interest:—

The arrangement of the book in its present form is the result of our having had constantly in view two matters of primary importance.

- 1 To give a selection of Compound Girders and Stanchions, sufficiently complete to obviate the making of a number of those minor calculations, which have been necessary when using any Section Book hitherto published.
2. To present the essential information respecting any Compound Section in one place to avoid the necessity of consulting different parts of the book for its composition and properties.

It will be found that our tables of Compound Girders and Stanchions are exceptionally comprehensive, and include all the forms in common use with a full range of plate thicknesses for every Standard Section sufficiently deep to be riveted.

As a further convenience the book is arranged in parts, each having a Contents page, and containing notes and formulæ explaining in detail the tables to which they refer.

A careful perusal of these notes should be made.

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Suggestions for details of construction and standardised connections are given in Part V., and, while these cannot always be rigidly adhered to, economies both in time and material may be effected in many cases by attention to them.

The properties and reference marks of the British Standard Sections of Steel Joists, Channels, Angles, and Tees have been taken, by permission, from the lists of the Engineering Standards Committee, and while these have been used (so far as applicable) in compiling the tables, the Engineering Standards Committee is in no way responsible for any of the figures which we publish.

All the tabulated results have been calculated and arranged by our Technical Department.

Safe loads are given in even figures to the nearest ton less, or, if the loads are small, to the nearest first decimal less, as we consider any greater degree of accuracy unnecessary.

In conclusion, we express the hope that this handbook may be found useful by all engaged in the design of Structural Steelwork, or otherwise interested in those materials or structures which we supply.

REDPATH, BROWN & CO., LTD.

### BROAD FLANGE BEAMS.

In addition to the British Standard Sections of Steel Joists we stock the following sections of Broad Flange Beams :—

14"	×	12"	×	164 lbs.	per foot.
12"	×	12"	×	82 lbs.	"
11"	×	11"	×	76 lbs.	"
10"	×	10"	×	61 lbs.	"
9"	×	9"	×	51 lbs.	"
8"	×	8"	×	44 lbs.	"
7"	×	7"	×	35 lbs.	"
6"	×	6"	×	23 lbs.	"
5½"	×	5½"	×	24 lbs.	"



**REDPATH, BROWN & CO., LIMITED.**

**PART I.**  
**GIRDERS.**

**SAFE LOADS**

**AND**

**PROPERTIES,**

**Etc.**



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# REDPATH, BROWN & CO., LIMITED.



## STEEL JOISTS.

Safe Distributed Loads, in Tons.

Reference Mark.	Size, D x B inches.	SPANS IN FEET.															
		10	12	14	16	18	20	22	24	26	28	30	32	36	40		
BSB 30	24 x 7 $\frac{1}{2}$			92	279	069	161	455	350	346	142	539	536	834	530	727	6
BSB 29	20 x 7 $\frac{1}{2}$	83	669	659	752	246	441	838	034	832	129	827	826	123	220	9	
BSB 28	18 x 7	63	853	245	639	035	531	929	026	624	522	821	319	917	715	9	
BSB 27	16 x 6	45	437	832	428	325	222	720	618	917	416	215	114	12	611	3	
BSB 26	15 x 6	11	934	929	926	223	320	919	017	416	114	913	913	011	6		
BSB 25	15 x 5	28	523	820	417	815	814	213	011	911	010	295	89	79			
BSB 24	14 x 6a	38	131	727	223	821	119	017	315	814	613	612	711	9			
BSB 23	14 x 6b	31	526	222	519	717	515	714	313	112	111	210	59	8			
BSB 22	12 x 6a	31	326	122	319	517	515	614	213	012	011	1					
BSB 21	12 x 6b	26	321	918	816	414	613	111	910	910	194						
BSB 20	12 x 5	18	315	313	111	410	291	83	76	70	65						
BSB 19	10 x 8	34	528	724	621	519	217	215	714	3							
BSB 18	10 x 6	21	117	615	113	211	710	596	88								
BSB 17	10 x 5	14	612	110	491	81	73	66	60								
BSB 16	9 x 7	25	521	317	915	914	212	711	6								

Tabular loads to right of zig-zag line will produce deflection greater than 1/26th of an inch per foot of span.

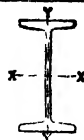
Let  $\delta$  = deflection in inches, K = deflection coefficient, and L = span in feet, then  $\delta = K \times L^2$ .

Safe working stress = 7.5 tons per square inch, equal to a factor of safety of 4. Ends of beams simply supported.

# REDPATH, BROWN & CO., LIMITED.

## STEEL JOISTS.

Dimensions and Properties.



Size, D x B inches.	Weight per foot in lbs.	Area in square inches	Standard Thicknesses.		Moments of Inertia.		Maximum Modulus of Section. X-X	Safe Dis- tributed Load on 1 foot Span.	Deflection Coefficient. X-X
			Web	Flange	Maxi- mum. X-X	Mini- mum. Y-Y			
24 x 7½	100	29.392	.600	1.070	2654.7	66.8	221.2	1106.1	.000781
20 x 7½	89	26.164	.600	1.010	1671.2	62.5	167.1	835.6	.000937
18 x 7	75	22.666	.550	.928	1149.6	46.6	127.7	638.7	.001041
16 x 6	62	18.227	.550	.847	725.9	27.0	90.7	453.7	.001172
15 x 6	59	17.346	.500	.880	629.0	28.2	83.8	419.4	.001250
15 x 5	42	12.351	.420	.647	428.2	11.9	57.1	285.4	.001250
14 x 6a	57	16.769	.500	.873	533.0	27.9	76.1	380.7	.001339
14 x 6b	46	13.533	.400	.698	440.6	21.5	62.9	314.7	.001339
12 x 6a	54	15.879	.500	.883	375.5	28.2	62.6	313.0	.001563
12 x 6b	44	12.916	.400	.717	315.4	22.2	52.5	262.8	.001563
12 x 5	32	9.408	.350	.550	229.1	9.7	36.6	183.4	.001563
10 x 8	70	20.582	.600	.970	345.0	71.6	69.0	345.0	.001875
10 x 6	42	12.358	.400	.736	211.6	22.9	42.3	211.6	.001875
10 x 5	30	8.820	.360	.552	145.6	9.7	29.1	145.6	.001875
9 x 7	58	17.064	.550	.924	229.7	46.2	51.0	255.2	.002083

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

All above sections are in our stocks.

For full explanations of tables, see notes commencing page 108.

For formulae, explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.



## STEEL JOISTS.

Safe Distributed Loads, in Tons.

Reference Mark.	Size, D x B inches.	SPANS IN FEET.																	
		3	4	5	6	7	8	9	10	11	12	14	16	18	20				
BSB 15	9 x 4				15.0	12.8	11.2	10.1	9.0	8.2	7.5	6.4	5.6	5.0	4.5				
BSB 14	8 x 6			27.3	23.0	19.7	17.3	15.3	13.8	12.6	11.5	9.8	8.6	7.7	6.9				
BSB 13	8 x 5				18.6	15.9	14.0	12.4	11.2	10.1	9.3	7.9	7.0	6.2	5.7				
BSB 12	8 x 4		17.4	13.9	11.6	9.9	8.6	7.7	6.9	6.3	5.8	4.9	4.3	3.8	3.4				
BSB 11	7 x 4			11.2	9.3	8.0	7.0	6.2	5.6	5.1	4.6	4.0	3.5	3.1					
BSB 10	6 x 5		18.2	14.5	12.1	10.3	9.0	8.1	7.2	6.6	6.0	5.1	4.5						
BSB 9	6 x 4½		14.4	11.5	9.6	8.2	7.2	6.4	5.8	5.2	4.8	4.1	3.6						
BSB 8	6 x 3	11.2	8.4	6.7	5.6	4.8	4.2	3.7	3.3	3.0	2.8	2.4	2.1						
BSB 7	5 x 4½			9.0	7.6	6.4	5.6	5.0	4.5	4.1	3.8								
BSB 6	5 x 3		6.8	5.4	4.5	3.9	3.4	3.0	2.7	2.4	2.2								
BSB 5	4½ x 1½	4.7	3.5	2.8	2.4	2.0	1.8	1.5	1.4	1.3	1.2								
BSB 4	4 x 3	6.2	4.7	3.7	3.1	2.5	2.3	2.1	1.9										
BSB 3	4 x 1½	3.0	2.3	1.8	1.5	1.3	1.1	1.0	0.9										
BSB 2	3 x 3	4.2	3.1	2.5	2.1	1.8	1.6												
BSB 1	3 x 1½	1.8	1.4	1.1	0.9	0.8	0.7												

Tabular loads to right of zigzag line will produce deflection greater than 1/20th of an inch per foot of span.

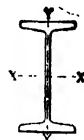
Let  $\delta$  = deflection in inches,  $K$  = deflection coefficient, and  $L$  = span in feet, then  $\delta = K \times L^2$ .

Safe working stress = 7.5 tons per square inch, equal to a factor of safety of 4. Ends of beams simply supported.

# REDPATH, BROWN & CO., LIMITED.

## STEEL JOISTS.

Dimensions and Properties.



Size, D x B inches.	Weight per foot in lbs.	Area in square inches.	Standard Thicknesses.		Moments of Inertia.		Maximum Modulus of Section. X-X	Safe Dis- tributed Load on 1 foot Span.	Deflection Coefficient. X-X
			Web.	Flange	Maxi- mum. X-X	Mini- mum. Y-Y			
9 x 4	21	6.178	.300	.460	81.1	4.2	18.0	90.1	.002083
8 x 6	35	10.293	.440	.597	110.5	17.9	27.6	138.2	.002344
8 x 5	28	8.241	.350	.575	89.3	10.2	22.3	111.7	.002344
8 x 4	18	5.297	.280	.402	55.7	3.5	13.9	69.6	.002344
7 x 4	16	4.709	.250	.387	39.2	3.4	11.2	56.0	.002679
6 x 5	25	7.354	.410	.520	43.6	9.1	14.5	72.7	.003125
6 x 4½	20	5.882	.370	.431	34.6	5.4	11.5	57.7	.003125
6 x 3	12	3.527	.260	.348	20.2	1.3	6.7	33.8	.003125
5 x 4½	18	5.290	.290	.448	22.6	5.6	9.1	45.4	.003750
5 x 3	11	3.238	.220	.376	13.6	1.4	5.4	27.2	.003750
4½ x 1½	6½	1.912	.180	.325	6.7	0.26	2.8	14.2	.003947
4 x 3	9½	2.795	.220	.336	7.5	1.3	3.7	18.8	.004688
4 x 1½	5	1.472	.170	.240	3.6	0.19	1.8	9.1	.004688
3 x 3	8½	2.501	.200	.332	3.7	1.2	2.5	12.6	.006250
3 x 1½	4	1.176	.160	.248	1.6	0.12	1.1	5.5	.006250

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

All above sections are in our stocks.

For full explanations of tables, see notes commencing page 108.

For formulae, explanations of properties, &c., see Part IV

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND GIRDERS.

Safe Distributed Loads, in Tons.

Reference Mark.	Size, D x B inches.	SPANS IN FEET.													
		14	16	18	20	22	24	26	28	30	32	34	36	40	44
296A	29 x 12												116	104	95.7
294A	28 1/2 x "											113	107	96.4	87.6
292A	28 x "									117	110	103	98.1	88.3	80.2
290A	27 1/2 x "								114	107	100	94.4	89.1	80.2	72.9
288A	27 x "						120	111	103	96.3	90.3	84.9	80.2	72.2	65.6
286A	26 1/2 x "					116	107	98.8	91.7	85.6	80.3	75.5	71.3	64.2	58.4
284A	26 x "			125	112	102	93.8	86.6	80.4	75.1	70.4	66.2	62.5	56.3	51.2
283A	25 3/4 x "		130	116	104	95.2	87.3	80.5	74.8	69.8	65.4	61.6	58.2	52.3	47.6
282A	25 1/2 x "		121	107	96.8	88.0	80.7	74.5	69.2	64.5	60.5	56.9	53.8	48.4	44.0
281A	25 1/4 x "	127	111	98.9	89.0	80.9	74.2	68.4	63.6	59.3	55.6	52.3	49.4	44.5	40.4
280A	25 x "	115	101	90.1	81.1	73.8	67.6	62.4	58.0	54.1	50.7	47.7	45.1	40.6	36.9
276A	25 x 12											100	94.8	85.3	77.6
274A	24 1/2 x "										98.0	92.5	87.1	78.4	71.3
272A	24 x "											98.0	92.5	87.1	78.4
270A	23 3/4 x "								102	95.4	89.5	84.2	79.5	71.6	65.1
268A	23 x "								99.7	92.6	86.4	81.0	76.2	72.0	64.8
266A	22 1/2 x "					105	96.9	89.4	83.0	77.5	72.6	68.3	64.5	58.1	52.8
264A	22 x "				102	93.5	85.8	79.2	73.5	68.6	64.3	60.5	57.2	51.4	46.8
263A	21 3/4 x "		112	99.6	89.7	81.5	74.7	69.0	64.0	59.8	56.0	52.7	49.8	44.8	40.7
262A	21 1/2 x "		103	92.3	83.1	75.5	69.2	63.9	59.3	55.4	51.9	48.9	46.1	41.5	37.8
261A	21 1/4 x "	109	95.7	85.0	76.5	69.6	63.8	58.9	54.7	51.0	47.8	45.0	42.5	38.3	34.8
260A	21 x "	100	87.5	77.8	70.0	63.6	58.3	53.9	50.0	46.7	43.8	41.2	38.9	35.0	31.8
260A	21 x "	90.7	79.4	70.6	63.5	57.7	52.9	48.9	45.4	42.3	39.7	37.4	35.3	31.7	28.8

Tabular loads to right of full zigzag line produce deflection greater than 1/26th of an inch per foot of span.

Girders supporting tabular loads to left of dotted zigzag line require stiffeners to prevent web buckling.

Girders supporting tabular loads printed in ordinary type have rivets at 6 inches pitch.

Girders supporting tabular loads printed in italics require a closer pitch of rivets. See page 50.

Safe working stress = 7.5 tons per square inch, equal to a factor of safety of 4. Ends of girders simply supported.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND GIRDERS.

### Composition and Properties.



Composed of		Weight per foot in lbs.	Area in square inches.	Maximum Moment of Inertia. x-x	Maximum Modulus of Section. x-x	Safe Distributed Load on 1 foot Span for		Deflection Coefficient. x-x
One Steel Joist.	Plates, each Flange to form.					Girder.	1 in Plate Width.	
24 x 7½	12 x 2½	308	89.4	12137	837.0	4185	303.5	.000646
"	" x 2½	287½	83.4	10994	771.5	3857	272.5	.000658
"	" x 2	267	77.4	9891	706.5	3532	242.0	.000670
"	" x 1½	246½	71.4	8826	641.9	3209	211.0	.000682
"	" x 1½	226	65.4	7799	577.7	2888	180.5	.000694
"	" x 1½	206	59.4	6810	513.9	2569	150.5	.000708
"	" x 1	185½	53.4	5857	450.5	2252	120.0	.000721
"	" x ¾	175	50.4	5394	418.9	2094	105.0	.000728
"	" x ¾	165	47.4	4940	387.4	1937	90.0	.000735
"	" x ¾	155	44.4	4495	356.0	1780	75.0	.000743
"	" x ¾	144½	41.4	4058	324.6	1623	60.0	.000750
20 x 7½	12 x 2½	297	86.2	8534	682.8	3414	254.0	.000750
"	" x 2½	276½	80.2	7687	627.5	3137	228.0	.000765
"	" x 2	256	74.2	6874	572.8	2864	202.0	.000781
"	" x 1½	235½	68.2	6094	518.6	2593	176.5	.000798
"	" x 1½	215	62.2	5347	464.9	2324	151.0	.000815
"	" x 1½	195	56.2	4631	411.6	2058	125.5	.000833
"	" x 1	174½	50.2	3946	358.7	1793	100.0	.000852
"	" x ¾	164	47.2	3615	332.4	1662	87.5	.000862
"	" x ¾	154	44.2	3292	306.2	1531	75.0	.000872
"	" x ¾	144	41.2	2976	280.1	1400	62.5	.000880
"	" x ¾	133½	38.2	2668	254.1	1270	50.0	.000893

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent over this must be allowed. See page 7.

Let  $C$  = deflection,  $K$  = deflection coefficient, and  $L$  = span in feet, then  $\delta = K \times L^3$ .

For full explanations of tables, see notes commencing page 108.

For formulae, explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND GIRDERS.

Safe Distributed Loads, in Tons.

Reference Mark.	Size, D x B inches.	SPANS IN FEET.													
		14	15	18	20	22	24	26	28	30	32	34	36	40	44
256A	23 x 12											87.8	82.9	74.6	67.8
254A	22 1/2 x "											80.3	75.9	68.3	62.1
252A	22 x "											82.8	77.6	70.0	62.1
250A	21 1/2 x "											86.0	79.9	71.6	65.0
248A	21 x "											83.0	76.7	68.3	61.7
246A	20 1/2 x "											84.0	77.6	69.0	62.1
244A	20 x "											84.0	77.6	69.0	62.1
243A	19 1/2 x "											87.0	77.3	69.0	62.1
242A	19 1/2 x "											91.0	79.9	71.6	65.0
241A	19 1/2 x "											82.5	72.2	64.2	57.8
240A	19 x "											74.1	64.8	57.6	51.9
232A	20 x 10														
230A	19 1/2 x "														
228A	19 x "														
226A	18 1/2 x "														
224A	18 x "														
223A	17 1/2 x "														
222A	17 1/2 x "														
221A	17 1/2 x "														
220A	17 x "														
212A	19 x 10														
210A	18 1/2 x "														
208A	18 x "														
206A	17 1/2 x "														
204A	17 x "														
203A	16 1/2 x "														
202A	16 1/2 x "														
201A	16 1/2 x "														
200A	16 x "														

Tabular loads to right of full zigzag line produce deflection greater than 1/25th of an inch per foot of span.

Girders supporting tabular loads to left of dotted zigzag line require stiffeners to prevent web buckling.

Girders supporting tabular loads printed in ordinary type have rivets at 6 inches pitch.

Girders supporting tabular loads printed in italics require a closer pitch of rivets. See page 51.

Safe working stress = 7.5 tons per square inch, equal to a factor of safety of 4. Ends of girders simply supported.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND GIRDERS.

### Composition and Properties.



Composed of		Weight per foot in lbs.	Area in square inches.	Maxi- mum Moment of Inertia. X-X	Maxi- mum Modulus of Section. X-X	Safe Distributed Load on 1 foot Span for		Deflection Coefficient. X-X
One Steel Joist.	Plates, each Flange to form.					Girder.	1 in Plate Width.	
18 x 7	12 x 2 1/2	283	82.1	6865	596.9	2984	229.5	.000815
"	" x 2 1/2	262 1/2	76.1	6149	546.5	2732	206.0	.000833
"	" x 2	242	70.1	5464	496.8	2484	182.5	.000852
"	" x 1 1/2	221 1/2	64.1	4810	447.4	2237	159.0	.000872
"	" x 1 1/2	201	58.1	4186	398.6	1993	136.0	.000893
"	" x 1 1/2	181	52.1	3590	350.2	1751	113.0	.000915
"	" x 1	160 1/2	46.1	3023	302.3	1511	90.0	.000938
"	" x 7/8	150	43.1	2750	278.4	1392	79.0	.000950
"	" x 7/8	140	40.1	2481	254.7	1273	67.5	.000961
"	" x 7/8	130	37.1	2224	231.1	1155	56.0	.000974
"	" x 7/8	119 1/2	34.1	1971	207.5	1037	45.0	.000987
16 x 6	10 x 2	200 1/2	58.2	3637	363.7	1818	162.5	.000938
"	" x 1 1/2	183 1/2	53.2	3189	327.1	1635	141.5	.000961
"	" x 1 1/2	166 1/2	48.2	2763	290.9	1454	121.0	.000987
"	" x 1 1/2	149 1/2	43.2	2360	255.1	1275	100.5	.001013
"	" x 1	132 1/2	38.2	1977	219.7	1098	80.5	.001041
"	" x 7/8	124	35.7	1794	202.1	1010	70.0	.001056
"	" x 7/8	115 1/2	33.2	1615	184.6	923	60.0	.001071
"	" x 7/8	107	30.7	1442	167.2	836	50.0	.001087
"	" x 7/8	98 1/2	28.2	1273	149.8	749	40.0	.001103
15 x 6	10 x 2	197 1/2	57.3	3226	339.6	1698	152.5	.000987
"	" x 1 1/2	180 1/2	52.3	2822	305.1	1525	133.0	.001013
"	" x 1 1/2	163 1/2	47.3	2440	271.1	1355	113.5	.001041
"	" x 1 1/2	146 1/2	42.3	2078	237.5	1187	94.5	.001071
"	" x 1	129 1/2	37.3	1736	204.8	1021	75.5	.001103
"	" x 7/8	121	34.8	1573	187.8	939	66.0	.001119
"	" x 7/8	112 1/2	32.3	1414	171.4	857	56.5	.001136
"	" x 7/8	101	29.8	1260	155.1	775	47.0	.001151
"	" x 7/8	95 1/2	27.3	1111	138.9	694	37.5	.001172

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2 1/2 per cent. over this must be allowed. See page 7

Let  $\delta$  = deflection,  $K$  = deflection coefficient, and  $L$  = span in feet, then  $\delta = K \times L^3$

For full explanations of tables see notes commencing page 103

For formula, explanations of properties, &c. see Part IV

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND GIRDERS.

Safe Distributed Loads, in Tons.

Reference Mark.	Size, D x B inches.	SPANS IN FEET.													
		10	12	14	16	18	20	22	24	26	28	30	32	34	36
188A	18 x 9	Rivets 3-in. diameter.													
186A	17 1/2 x "	56-9-51-7-47-4-43-8-40-6-37-9-35-6-33-5-31-6													
184A	17 x "	54-9-49-6-44-9-41-4-38-0-35-2-32-9-30-9-29-0-27-4													
183A	16 1/2 x "	63-6-54-5-47-7-42-4-38-1-34-7-31-9-29-3-27-2-25-4-23-8-22-4-21-2													
182A	16 1/4 x "	57-4-49-2-43-0-38-2-34-4-31-3-28-7-26-5-24-6-22-9-21-5-20-2-19-1													
181A	16 1/2 x "	61-5-51-2-43-9-38-4-34-1-30-7-27-9-25-6-23-6-22-0-20-5-19-2-18-1-17-1													
180A	16 x "	54-1-45-1-38-7-33-8-30-1-27-1-24-6-22-6-20-8-19-3-18-0-16-9-15-9-15-0													
179A	15 1/2 x "	46-8-39-0-33-4-29-2-26-0-23-4-21-3-19-5-18-0-16-7-15-6-14-6-13-7													
172A	18 x 10	60-6-56-5-52-5-49-2-46-3-43-8													
170A	17 1/2 x "	58-9-54-4-50-5-47-1-44-2-41-6-39-3													
168A	17 x "	62-7-57-6-53-3-48-3-44-8-41-8-39-2-36-9-34-8													
166A	16 1/2 x "	61-0-54-2-49-9-45-7-42-2-39-2-36-6-34-3-32-3-30-5													
164A	16 x "	67-3-58-9-53-4-47-1-42-8-39-3-36-3-33-7-31-4-29-4-27-7-26-2													
163A	15 1/2 x "	72-1-61-8-54-7-48-1-43-3-36-3-36-1-33-3-30-9-28-8-27-0-25-5													
162A	15 1/4 x "	65-8-56-4-49-3-43-8-39-4-35-9-32-9-30-3-28-2-26-3-24-6-23-2													
161A	15 1/2 x "	71-3-59-4-50-9-44-6-39-6-35-6-32-4-29-7-27-4-25-5-23-8-22-3-21-0													
160A	15 x "	63-7-53-1-45-5-39-8-35-4-31-8-28-9-26-5-24-5-22-7-21-2-19-9-18-7													
159A	14 1/2 x "	56-2-46-8-40-1-35-1-31-2-28-1-25-5-23-4-21-6-20-1-18-7-17-5													
148A	17 x 10	46-4-43-7-40-2-37-7-35-5-33-5													
146A	16 1/2 x "	47-7-43-7-40-3-37-1-34-3-31-8-29-7-27-8-26-2-24-8													
144A	16 x "	49-5-44-6-40-5-37-1-34-3-31-8-29-7-27-8-26-2-24-8													
143A	15 1/2 x "	50-8-45-2-40-7-37-0-33-9-31-3-29-0-27-1-25-4-23-9													
142A	15 1/4 x "	52-6-46-0-40-9-36-8-33-5-30-7-28-3-26-3-24-5-23-0-21-7													
141A	15 1/2 x "	55-0-47-1-41-2-36-6-33-0-30-0-27-5-25-4-23-5-22-0-20-6-19-4													
140A	15 x "	48-6-41-6-36-4-32-4-29-1-26-5-24-3-22-4-20-8-19-4-18-2-17-1													
139A	14 1/2 x "	50-7-42-2-36-2-31-7-28-2-25-3-23-0-21-7-19-5-18-1-16-9-15-8													

Tabular loads to right of full zigzag line produce deflection greater than 1/26th of an inch per foot of span

Girders supporting tabular loads to left of dotted zigzag line require stiffeners to prevent web buckling

Girders supporting tabular loads printed in ordinary type have rivets at 6 inches pitch

Girders supporting tabular loads printed in italics require a closer pitch of rivets See page 51

Safe working stress = 7.5 tons per square inch, equal to a factor of safety of 4 Ends of girders simply supported

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND GIRDERS.

Composition and Properties.



Composed of		Weight per foot in lbs.	Area in square inches.	Maximum Moment of Inertia. X-X	Maximum Modulus of Section. X-X	Safe Distributed Load on 1 foot Span for		Deflection Coefficient. X-X
One Steel Joist.	Plates, each Flange to form.					Girder.	1 in. Plate Width.	
15 x 5	9 x 1 1/2	136 1/2	39.4	2050	227.8	1139	113.5	.001041
"	" x 1 1/2	121	34.9	1728	197.5	987	94.5	.001071
"	" x 1	105 1/2	30.4	1423	167.5	837	75.5	.001103
"	" x 7/8	98	28.1	1278	152.6	763	66.0	.001119
"	" x 3/4	90 1/2	25.9	1136	137.7	688	56.5	.001136
"	" x 5/8	82 1/2	23.6	999	123.0	615	47.0	.001151
"	" x 1/2	75	21.4	866	108.3	541	37.5	.001172
"	" x 3/8	67 1/2	19.1	737	93.6	468	28.0	.001190
14 x 6a	10 x 2	195 1/2	56.8	2837	315.2	1576	143.0	.001041
"	" x 1 1/2	178 1/2	51.8	2475	282.9	1414	124.5	.001071
"	" x 1 1/4	161 1/2	46.8	2133	251.0	1255	106.0	.001103
"	" x 1 1/2	144 1/2	41.8	1811	219.5	1097	88.0	.001136
"	" x 1	127 1/2	37.8	1508	188.5	942	70.5	.001172
"	" x 7/8	119	34.3	1363	173.1	865	61.5	.001190
"	" x 3/4	110 1/2	31.8	1223	157.8	789	52.5	.001210
"	" x 5/8	102	29.3	1087	142.6	713	44.0	.001230
"	" x 1/2	93 1/2	26.8	956	127.4	637	35.0	.001250
"	" x 3/8	85	24.3	829	112.4	562	26.0	.001271
14 x 6b	10 x 1 1/2	150 1/2	43.5	2052	241.4	1207	106.0	.001103
"	" x 1 1/4	133 1/2	38.5	1730	209.7	1048	88.0	.001136
"	" x 1	116 1/2	33.5	1427	178.3	891	70.5	.001172
"	" x 7/8	108	31.0	1282	162.8	814	61.5	.001190
"	" x 3/4	99 1/2	28.5	1142	147.3	736	52.5	.001210
"	" x 5/8	91	26.0	1006	131.9	659	44.0	.001230
"	" x 1/2	82 1/2	23.5	875	116.6	583	35.0	.001250
"	" x 3/8	74	21.0	748	101.4	507	26.0	.001271

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 3 1/4 per cent. over this must be allowed. See page 7.

Let  $\delta$  = deflection,  $K$  = deflection coefficient, and  $L$  = span in feet, then  $\delta = K \times L^4$ .

For full explanation of tables, see notes commencing page 103.

For formulae, explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND GIRDERS.

Safe Distributed Loads, in Tons.

Reference Mark.	Size, D x B inches.	SPANS IN FEET.													
		8	10	12	14	16	18	20	22	24	26	28	30	32	34
132A	16 x 10										51.6	47.9	44.7	41.9	39.4
130A	15 1/2 x "									54.5	50.9	46.2	42.9	40.0	37.6
128A	15 x "									53.1	48.3	44.2	40.8	37.7	35.3
126A	14 1/2 x "									51.6	46.8	42.7	39.3	36.2	33.2
124A	14 x "									50.6	45.8	41.7	38.3	35.2	32.4
123A	13 1/2 x "									49.7	44.9	40.8	37.4	34.3	31.4
122A	13 1/2 x "									48.7	43.9	39.8	36.4	33.3	30.4
121A	13 1/2 x "									47.7	42.9	38.8	35.4	32.3	29.4
120A	13 x "									46.7	41.9	37.8	34.4	31.3	28.4
119A	12 1/2 x "									45.7	40.9	36.8	33.4	30.3	27.4
108A	15 x 10										39.5	36.7	34.2	32.1	30.2
106A	14 1/2 x "										40.6	37.8	35.3	33.1	31.0
104A	14 x "										41.9	37.7	34.3	31.9	29.8
103A	13 1/2 x "										43.0	38.3	34.4	31.9	29.6
102A	13 1/2 x "										43.8	38.6	34.4	31.9	29.6
101A	13 1/2 x "										44.4	38.9	34.5	31.9	29.6
100A	13 x "										46.4	39.7	34.8	30.9	27.8
99A	12 1/2 x "										49.1	40.9	35.1	30.7	27.5
88A	15 x 9										36.5	32.7	30.2	27.4	25.8
86A	14 1/2 x "										37.8	33.4	30.7	27.9	25.9
84A	14 x "										39.5	35.1	32.3	29.5	27.5
83A	13 1/2 x "										35.7	31.7	28.6	25.9	23.6
82A	13 1/2 x "										35.7	31.7	28.6	25.9	23.6
81A	13 1/2 x "										36.5	32.2	28.8	25.9	23.6
80A	13 x "										37.7	32.8	29.2	26.2	23.7
79A	12 1/2 x "										39.3	32.7	28.1	24.6	21.8

Tabular loads to right of full zigzag line produce deflection greater than 1/26th of an inch per foot of span.

Girders supporting tabular loads to left of dotted zigzag line require stiffeners to prevent web buckling.

Girders supporting tabular loads printed in ordinary type have rivets at 6 inches pitch.

Girders supporting tabular loads printed in italics require a closer pitch of rivets. See page 52.

Safe working stress = 7.5 tons per square inch, equal to a factor of safety of 4. Ends of girders simply supported.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND GIRDERS.

Composition and Properties.



Composed of		Weight per foot in lbs.	Area in square inches.	Maximum Moment of Inertia. X-X	Maximum Modulus of Section. X-X	Safe Distributed Load on 1 foot Span for		Deflection Coefficient. X-X
One Steel Joist.	Plates, each Flange to form.					Girder.	1 in. Plate Width.	
12 x 6a	10 x 2	192½	55.9	2145	268.1	1340	123.0	.001172
"	" x 1½	175½	50.9	1860	240.0	1260	107.0	.001210
"	" x 1½	158½	45.9	1593	212.4	1062	91.5	.001250
"	" x 1¼	141½	40.9	1347	185.8	929	76.0	.001293
"	" x 1	125	35.9	1110	158.5	792	60.5	.001339
"	" x ¾	116	33.4	999	145.3	726	52.5	.001364
"	" x ¾	107½	30.9	892	132.2	661	45.0	.001389
"	" x ¾	99	28.4	790	119.2	596	37.5	.001415
"	" x ¾	90½	25.9	691	106.3	531	30.0	.001442
"	" x ¾	82	23.4	596	93.4	467	22.5	.001471
12 x 6b	10 x 1½	148½	42.9	1540	205.3	1026	91.5	.001250
"	" x 1½	131½	37.9	1294	178.6	893	76.0	.001293
"	" x 1	114½	32.9	1057	151.0	755	60.5	.001339
"	" x ¾	106	30.4	916	137.7	688	52.5	.001364
"	" x ¾	97½	27.9	810	124.4	622	45.0	.001389
"	" x ¾	89	25.4	737	111.3	556	37.5	.001415
"	" x ¾	80½	22.9	638	98.2	491	30.0	.001442
"	" x ¾	72	20.4	543	85.2	426	22.5	.001471
12 x 5	9 x 1½	126½	36.4	1315	175.3	876	91.5	.001250
"	" x 1½	111	31.9	1096	151.2	756	76.0	.001293
"	" x 1	95½	27.4	884	126.3	631	60.5	.001339
"	" x ¾	88	25.2	785	114.3	571	52.5	.001364
"	" x ¾	80½	22.9	690	102.3	511	45.0	.001389
"	" x ¾	72½	20.7	599	90.4	452	37.5	.001415
"	" x ¾	65	18.4	511	78.6	393	30.0	.001442
"	" x ¾	57½	16.2	426	66.8	334	22.5	.001471

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

Let  $\delta$  = deflection,  $K$  = deflection coefficient, and  $L$  = span in feet, then  $\delta = K \times L^3$ .

For full explanations of tables, see notes commencing page 108.

For formulae, explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND GIRDERS.

Safe Distributed Loads, in Tons.

Reference Mark.	Size, D x B inches.	SPANS IN FEET.												
		6	8	10	12	14	16	18	20	22	24	26	28	30
68A	13 x 10										35.3	32.6	30.3	28.3
66A	12½ x "									33.3	30.5	28.2	26.2	
64A	12 x "								34.4	31.0	28.2	25.8	23.8	22.1
63A	11½ x "								35.2	31.3	28.2	25.6	23.5	21.7
62A	11¼ x "								36.3	31.8	28.3	25.4	23.1	21.2
61A	11¼ x "								37.8	32.4	28.4	25.2	22.7	20.6
60A	11 x "								40.0	33.3	28.6	25.0	22.2	20.0
59A	10½ x "								43.2	34.6	28.8	24.7	21.6	19.2
48A	13 x 9											30.2	27.9	25.9
46A	12½ x "										31.0	28.2	25.9	23.9
44A	12 x "										32.4	28.8	25.9	23.6
43A	11½ x "										33.4	29.3	26.0	23.4
42A	11¼ x "										34.9	29.9	26.1	23.2
41A	11¼ x "										36.9	30.7	26.4	23.1
40A	11 x "										40.0	31.9	26.6	22.8
39A	10½ x "										45.1	33.9	27.1	22.6
28A	11 x 10										33.9	29.9	27.4	25.3
26A	10½ x "										31.9	28.2	25.6	23.5
24A	10 x "										33.6	29.4	26.2	23.5
23A	9½ x "										35.5	30.4	26.6	23.7
22A	9¼ x "										38.1	31.7	27.3	23.8
21A	9¼ x "										33.7	28.7	24.1	21.1
20A	9 x "										36.6	29.3	24.4	20.9
19A	8½ x "										41.6	31.2	24.9	20.8
14A	10 x 9													
13A	9½ x "													
12A	9¼ x "													
11A	9¼ x "													
10A	9 x "													
9A	8½ x "													

Tabular loads to right of full zigzag line produce deflection greater than 1/26th of an inch per foot of span

Girders supporting tabular loads to left of dotted zigzag line require stiffeners to prevent web buckling.

Girders supporting tabular loads printed in ordinary type have rivets at 6 inches pitch.

Girders supporting tabular loads printed in italics require a closer pitch of rivets. See page 52

Safe working stress = 7.5 tons per square inch, equal to a factor of safety of 4. Ends of girders simply supported

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND GIRDERS.

Composition and Properties.



Composed of		Weight per foot in lbs.	Area in square inches.	Maxi- mum Moment of Inertia. x-x	Maxi- mum Modulus of Section. x-x	Safe Distributed Load on 1 foot Span for		Deflection Coefficient. x-x
One Steel Joist.	Plates, each Flange to form.					Girder.	1 in. Plate Width.	
10 x 6	10 x 1½	146½	42.4	1103	169.6	848	76.5	.001442
"	" x 1½	129½	37.4	916	146.6	733	63.5	.001500
"	" x 1	112½	32.4	744	124.0	620	50.5	.001563
"	" x ¾	104	29.9	663	112.8	564	44.0	.001596
"	" x ¾	95½	27.4	585	101.8	509	37.5	.001630
"	" x ¾	87	24.9	511	90.8	454	31.5	.001667
"	" x ¾	78½	22.4	440	80.0	400	25.0	.001704
"	" x ¾	70	19.9	372	69.2	346	18.5	.001744
10 x 5	9 x 1½	124½	35.8	942	145.0	725	76.5	.001442
"	" x 1½	109	31.3	776	124.2	621	63.5	.001500
"	" x 1	93½	26.8	622	103.8	519	50.5	.001563
"	" x ¾	86	24.6	550	93.7	468	44.0	.001596
"	" x ¾	78½	22.3	481	83.7	418	37.5	.001630
"	" x ¾	70½	20.1	415	73.8	369	31.5	.001667
"	" x ¾	63	17.8	351	63.9	319	25.0	.001704
"	" x ¾	55½	15.6	291	54.2	271	18.5	.001744
8 x 6	10 x 1½	139½	40.3	724	131.7	659	62.0	.001704
"	" x 1½	122½	35.3	592	112.7	563	51.0	.001785
"	" x 1	105½	30.3	471	94.2	471	40.5	.001875
"	" x ¾	97	27.8	415	85.2	426	35.5	.001923
"	" x ¾	88½	25.3	362	76.2	381	30.0	.001974
"	" x ¾	80	22.8	311	67.4	337	25.0	.002027
"	" x ¾	71½	20.3	263	58.6	293	20.0	.002083
"	" x ¾	63	17.8	218	49.9	249	15.0	.002143
8 x 5	9 x 1	91½	26.2	409	81.9	409	40.5	.001875
"	" x ¾	83½	24.0	359	73.7	368	35.5	.001923
"	" x ¾	76½	21.7	312	65.7	328	30.0	.001974
"	" x ¾	69	19.5	267	57.8	289	25.0	.002027
"	" x ¾	61	17.2	224	49.9	249	20.0	.002083
"	" x ¾	53½	15.0	184	42.1	210	15.0	.002143

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

Let  $\delta$  = deflection,  $K$  = deflection coefficient, and  $L$  = span in feet, then  $\delta = K \times L^3$ .

For full explanations of tables, see notes commencing page 108.

For formulae, explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND GIRDERS.

Safe Distributed Loads, in Tons.

Reference Mark.	Size, D x B inches.	SPANS IN FEET.															
		14	16	18	20	22	24	26	28	30	32	34	36	40	44		
296B	29 x 16									198	185	174	165	148	135		
294B	28½ x "									197	184	172	162	153	138	125	
292B	28 x "									213	197	183	171	160	142	128	116
290B	27½ x "									197	181	168	157	147	134	131	118
288B	27 x "									168	154	141	135	127	120	108	98.4
286B	26½ x "									151	140	131	123	115	109	98.4	89.4
284B	26 x "									151	140	131	123	115	109	98.4	89.4
283B	25½ x "									154	141	135	127	120	108	98.4	89.4
282B	25½ x "									154	141	135	127	120	108	98.4	89.4
281B	25½ x "									154	141	135	127	120	108	98.4	89.4
280B	25 x "									154	141	135	127	120	108	98.4	89.4
276B	25 x 16									159	144	131	123	114	109		
274B	24½ x "									158	148	136	130	123	111	101	
272B	24 x "									158	148	136	130	123	111	101	
270B	23½ x "									171	158	146	137	128	120	114	102
268B	23 x "									171	158	146	137	128	120	114	102
266B	22½ x "									171	158	146	137	128	120	114	102
264B	22 x "									171	158	146	137	128	120	114	102
263B	21½ x "									171	158	146	137	128	120	114	102
262B	21½ x "									171	158	146	137	128	120	114	102
261B	21½ x "									171	158	146	137	128	120	114	102
260B	21 x "									171	158	146	137	128	120	114	102

Tabular loads to right of full zigzag line produce deflection greater than 1/26th of an inch per foot of span.  
 Girders supporting tabular loads to left of dotted zigzag line require stiffeners to prevent web buckling.  
 Girders supporting tabular loads printed in ordinary type have rivets at 6 inches pitch.  
 Girders supporting tabular loads printed in italics require a closer pitch of rivets. See page 53.  
 Safe working stress = 7.5 tons per square inch, equal to a factor of safety of 4. Ends of girders simply supported.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND GIRDERS.

### Composition and Properties.



Composed of		Weight per foot in lbs.	Area in square inches.	Maxi- mum Moment of Inertia. X-X	Maxi- mum Modulus of Section. X-X	Safe Distributed Load on 1 foot Span for		Deflection Coefficient. X-X
Two steel Joists.	Plates, each Flange to form.					Girder.	1 in. Plate Width.	
24 x 7½	16 x 2½	475½	138.7	17232	1188.4	5942	303.5	.000646
"	" x 2½	448½	130.7	15772	1106.4	5534	272.5	.000658
"	" x 2	421½	122.7	14363	1025.9	5129	242.0	.000669
"	" x 1½	394	114.7	13001	945.7	4728	211.0	.000681
"	" x 1½	367	106.7	11692	866.1	4330	180.5	.000694
"	" x 1½	339½	98.7	10129	787.1	3935	150.5	.000707
"	" x 1	312½	90.7	9212	708.6	3543	120.0	.000721
"	" x ¾	299	86.7	8621	669.6	3348	105.0	.000728
"	" x ¾	285½	82.7	8012	630.7	3153	90.0	.000735
"	" x ¾	271	78.7	7473	591.9	2959	75.0	.000742
"	" x ¾	258	74.7	6916	553.3	2766	60.0	.000750
20 x 7½	16 x 2½	453½	132.5	11985	958.9	4794	254.0	.000750
"	" x 2½	426½	124.5	10901	890.1	4450	228.0	.000765
"	" x 2	399½	116.5	9866	822.2	4111	202.0	.000781
"	" x 1½	372	108.5	8870	754.8	3774	176.5	.000797
"	" x 1½	345	100.5	7915	688.3	3441	151.0	.000815
"	" x 1½	317½	92.5	7001	622.4	3112	125.5	.000833
"	" x 1	290½	84.5	6127	557.0	2785	100.0	.000852
"	" x ¾	277	80.5	5705	524.6	2623	87.5	.000862
"	" x ¾	263½	76.5	5292	492.2	2461	75.0	.000872
"	" x ¾	249½	72.5	4889	460.1	2300	62.5	.000880
"	" x ¾	236	68.5	4495	428.1	2140	50.0	.000893

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

Let  $\delta$  = deflection,  $K$  = deflection coefficient, and  $L$  = span in feet, then  $\delta = K \times L^4$ .

For full explanations of tables, see notes commencing page 108.

For formulae, explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND GIRDERS.

Safe Distributed Loads, in Tons.

Reference Mark.	Size, D x B inches.	SPANS IN FEET.													
		14	16	18	20	22	24	26	28	30	32	34	36	40	44
256B	23 x 16										129	121	114	103	93.9
254B	22½ x "									126	127	119	112	106	95.4
252B	22 x "									125	116	109	103	97.4	87.7
250B	21½ x "									123	114	106	100	94.1	88.9
248B	21 x "									121	111	103	96.6	90.5	85.2
246B	20½ x "									120	110	102	94.8	88.7	83.4
244B	20 x "	185	162	144	129	118	108	99.9	92.8	86.6	81.2	76.4	72.1	64.9	59.0
243B	19½ x "	164	143	127	115	104	95.8	88.5	82.1	76.7	71.9	67.6	63.3	57.5	52.3
242B	19 x "	153	134	119	107	97.8	89.7	82.8	76.9	71.8	67.3	63.3	59.8	53.8	48.9
241B	19½ x "	143	125	111	100	91.2	83.6	77.2	71.6	66.9	62.7	59.0	55.7	50.1	45.6
240B	19 x "	132	116	103	93.0	84.5	77.5	71.6	66.6	62.0	58.1	54.7	51.7	46.5	42.3
		122	107	95.3	85.7	77.9	71.4	66.0	61.2	57.2	53.6	50.4	47.6	42.9	38.9
232B	20 x 14										88.7	83.1	78.2	73.9	66.5
230B	19½ x "										87.1	81.6	76.7	72.6	65.0
228B	19 x "										85.9	80.4	75.6	71.5	64.0
226B	18½ x "										84.7	79.2	74.4	70.3	62.8
224B	18 x "	122	107	95.2	85.7	77.9	71.4	66.0	61.2	57.2	53.6	50.4	47.6	42.9	38.9
223B	17½ x "	114	99.9	88.8	80.0	72.7	66.6	61.5	57.1	53.3	50.0	47.0	44.4	40.0	
222B	17 x "	106	92.8	82.5	74.2	67.5	61.9	57.1	53.0	49.5	46.4	43.7	41.2	37.1	
221B	17½ x "	98.0	85.7	76.2	68.6	62.3	57.1	52.7	49.0	45.7	42.9	40.3	38.1	34.3	
220B	17 x "	89.9	78.6	69.9	62.9	57.2	52.4	48.4	44.9	41.9	39.3	37.0	34.9	31.5	
212B	19 x 14										82.6	77.5	72.9	68.9	62.0
210B	18½ x "										81.0	76.0	71.4	67.4	61.2
208B	18 x "										79.4	74.4	69.8	65.8	59.6
206B	17½ x "										77.8	72.8	68.2	64.2	58.0
204B	17 x "	113	99.4	88.3	79.5	72.3	66.3	61.2	56.8	53.0	49.7	46.8	44.2	39.1	
203B	16½ x "	105	92.7	82.4	74.1	67.4	61.8	57.0	52.9	49.4	46.3	43.6	41.2		
202B	16 x "	98.3	86.0	76.4	68.8	62.5	57.3	52.9	49.1	45.8	43.0	40.4	38.2		
201B	16½ x "	90.6	79.3	70.5	63.4	57.7	52.9	48.8	45.3	42.3	39.6	37.3	35.2		
200B	16 x "	83.1	72.7	64.6	58.2	52.9	48.5	44.7	41.5	38.8	36.3	34.2	32.8		

Tabular loads to right of full zigzag line produce deflection greater than 1/26th of an inch per foot of span.

Girders supporting tabular loads to left of dotted zigzag line require stiffeners to prevent web buckling.

Girders supporting tabular loads printed in ordinary type have rivets at 6 inches pitch.

Girders supporting tabular loads printed in italics require a closer pitch of rivets. See page 54.

Safe working stress = 7.5 tons per square inch, equal to a factor of safety of 4. Ends of girders simply supported

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND GIRDERS.

### Composition and Properties.



Composed of		Weight per foot in lbs.	Area in square inches.	Maxi- mum Moment of Inertia. X-X	Maxi- mum Modulus of Section. X-X	Safe Distributed Load on 1 foot Span for		Deflection Coefficient. X-X
Two* Steel Joists.	Plates, each Flange to form.					Girder.	1 in. Plate Width.	
18 x 7	16 x 2½	426	124.1	9506	826.6	4133	229.5	.000815
"	" x 2½	398½	116.1	8592	763.7	3818	206.0	.000833
"	" x 2	371½	108.1	7718	701.6	3508	182.5	.000852
"	" x 1½	344½	100.1	6883	640.3	3201	159.0	.000872
"	" x 1½	317	92.1	6085	579.6	2898	136.0	.000893
"	" x 1½	290	84.1	5325	519.5	2597	113.0	.000914
"	" x 1	262½	76.1	4601	460.1	2300	90.0	.000937
"	" x 7/8	249	72.1	4252	430.6	2153	79.0	.000949
"	" x 3/4	235½	68.1	3912	401.3	2006	67.5	.000961
"	" x 3/4	222	64.1	3581	372.0	1860	56.5	.000974
"	" x 3/4	208½	60.1	3258	343.0	1715	45.0	.000987
16 x 6	14 x 2	317	92.4	5322	532.2	2661	162.5	.000937
"	" x 1½	293	85.4	4719	484.0	2420	141.5	.000961
"	" x 1½	269	78.4	4145	436.4	2182	121.0	.000987
"	" x 1½	245½	71.4	3602	389.4	1947	100.5	.001013
"	" x 1	221½	64.4	3086	342.9	1714	80.5	.001041
"	" x 7/8	209½	60.9	2839	319.9	1599	70.0	.001066
"	" x 3/4	198	57.4	2599	297.0	1485	60.0	.001071
"	" x 3/4	186	53.9	2366	274.3	1371	50.0	.001087
"	" x 3/4	174	50.4	2139	251.6	1258	40.0	.001108
15 x 6	14 x 2	311	90.6	4711	495.9	2479	152.5	.000987
"	" x 1½	287	83.6	4167	450.5	2252	133.0	.001013
"	" x 1½	263½	76.6	3652	405.7	2028	113.5	.001041
"	" x 1½	239½	69.6	3164	361.6	1808	94.5	.001071
"	" x 1	215½	62.6	2704	318.1	1590	75.5	.001103
"	" x 7/8	204	59.1	2484	296.5	1482	66.0	.001119
"	" x 3/4	192	55.6	2270	275.1	1375	56.5	.001136
"	" x 3/4	180	52.1	2062	253.8	1269	47.0	.001151
"	" x 3/4	168	48.6	1861	232.7	1163	37.5	.001172

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

Let  $\delta$  = deflection,  $K$  = deflection coefficient, and  $L$  = span in feet, then  $\delta = K \times L^4$ .

For full explanations of tables, see notes commencing page 108.

For formulae, explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND GIRDERS.

Safe Distributed Loads, in Tons.

Reference Mark.	Size, D x B inches.	SPANS IN FEET.															
		10	12	14	16	18	20	22	24	26	28	30	32	34	36		
188B	18 x 12	Rivets 3-in. diameter.															
186B	17½ x "	88.0	78.2	70.4	64.0	58.7	54.1	50.3	46.9	44.0	41.4	39.1	36.9	34.9	33.1		
184B	17 x "	101.8	87.4	76.4	67.9	61.1	55.5	50.9	47.0	43.7	40.7	38.2	35.9	33.9			
183B	16½ x "	113.9	97.2	80.8	70.7	62.8	56.5	51.4	47.1	43.5	40.4	37.7	35.3	33.2	31.4		
182B	16 x "	103.8	86.0	74.2	64.9	57.7	51.9	47.2	43.3	40.0	37.1	34.6	32.5	30.5	28.8		
181B	16½ x "	94.8	79.0	67.7	59.2	52.6	47.4	43.1	39.5	36.5	33.8	31.6	29.6	27.9	26.3		
180B	16 x "	85.8	71.4	61.2	53.6	47.6	42.9	39.0	35.7	33.0	30.6	28.6	26.8	25.2	23.8		
179B	15½ x "	76.7	63.9	54.8	47.9	42.6	38.3	34.9	31.9	29.4	27.4	25.6	24.0	22.5			
172B	18 x 14	Rivets 3-in. diameter.															
170B	17½ x "								80.1	74.3	69.4	65.0	61.2	57.8			
168B	17 x "							85.1	78.0	72.0	66.9	62.4	58.2	55.1	52.0		
166B	16½ x "					92.6	83.3	75.7	69.4	64.1	59.5	55.5	52.0	49.0	46.2		
164B	16 x "	104.1	91.4	81.2	73.1	66.5	60.9	56.2	52.4	48.7	45.7	43.0	40.6				
163B	15½ x "	113.9	97.3	85.1	75.7	68.1	61.9	56.8	52.4	48.6	45.4	42.6	40.1				
162B	15½ x "	126.1	105.9	90.2	78.9	70.1	63.1	57.4	52.6	48.5	45.1	42.1	39.4	37.1			
161B	15½ x "	116.9	96.9	83.1	72.7	64.6	58.1	52.8	48.4	44.7	41.5	38.8	36.3	34.2			
160B	15 x "	106.8	88.7	76.0	66.5	59.1	53.2	48.4	44.3	40.9	38.0	35.5	33.3	31.3			
159B	14½ x "	96.7	80.5	69.0	60.4	53.7	48.3	43.9	40.3	37.2	34.5	32.2	30.2				
148B	17 x 14	Rivets 3-in. diameter.															
146B	16½ x "					87.1	78.4	71.2	65.3	60.3	56.0	52.2	49.0	46.7	43.5		
144B	16 x "					85.1	75.6	68.0	61.9	56.7	52.4	48.6	45.4	42.5	40.3		
143B	15½ x "	105.8	89.2	78.7	69.9	62.9	57.7	52.5	48.4	44.5	40.9	38.3	37.0				
142B	15½ x "	115.9	96.5	82.7	72.4	64.3	57.9	52.6	48.2	44.5	41.3	38.6	36.2	34.0			
141B	15½ x "	105.8	88.0	75.5	66.0	58.7	52.8	48.0	44.0	40.6	37.7	35.2	33.0	31.1			
140B	15 x "	95.6	79.7	68.3	59.7	53.1	47.8	43.5	39.8	36.8	34.1	31.9	29.9	28.1			
139B	14½ x "	85.7	71.1	61.1	53.5	47.5	42.8	38.9	35.7	32.9	30.6	28.5	26.7				

Tabular loads to right of full zigzag line produce deflection greater than 1/26th of an inch per foot of span.

Girders supporting tabular loads to left of dotted zigzag line require stiffeners to prevent web bulking.

Girders supporting tabular loads printed in ordinary type have rivets at 6 inches pitch.

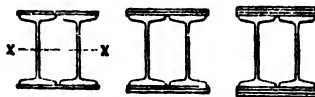
Girders supporting tabular loads printed in italics require a closer pitch of rivets. See page 54.

Safe working stress = 7.5 tons per square inch, equal to a factor of safety of 4. Ends of girders simply supported.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND GIRDERS.

### Composition and Properties.



Composed of		Weight per foot in lbs.	Area in square inches.	Maxi- mum Moment of Inertia. x-x	Maxi- mum Modulus of Section. x-x	Safe Distributed Load on 1 foot Span for		Deflection Coefficient. x-x
Two Steel Joists.	Plates, each Flange to form.					Girder.	1 in. Plate Width.	
15 x 5	12 x 1 1/2	209	60.7	2873	319.2	1596	113.5	.001041
"	" x 1 1/2	188 1/2	54.7	2464	281.6	1408	94.5	.001071
"	" x 1	168	48.7	2078	244.5	1222	75.5	.001103
"	" x 7/8	158	45.7	1893	226.1	1130	66.0	.001119
"	" x 3/4	147 1/2	42.7	1714	207.8	1039	56.5	.001136
"	" x 5/8	137 1/2	39.7	1540	189.6	948	47.0	.001151
"	" x 1/2	127 1/2	36.7	1372	171.5	857	37.5	.001172
"	" x 3/8	117	33.7	1208	153.4	767	28.0	.001190
14 x 6a	14 x 2	307	89.5	4130	458.9	2294	143.0	.001041
"	" x 1 1/2	283	82.5	3643	416.3	2081	124.5	.001071
"	" x 1 1/4	259 1/2	75.5	3182	374.4	1872	106.0	.001103
"	" x 1 1/2	235 1/2	68.5	2748	333.1	1665	88.0	.001136
"	" x 1	211 1/2	61.5	2340	292.5	1462	70.5	.001172
"	" x 7/8	200	58.0	2145	272.4	1362	61.5	.001190
"	" x 3/4	188	54.5	1956	252.4	1262	52.5	.001210
"	" x 1/2	176	51.0	1773	232.6	1163	44.0	.001230
"	" x 5/8	164	47.5	1596	212.9	1064	35.0	.001250
"	" x 3/8	152	44.0	1425	193.2	966	26.0	.001271
14 x 6b	14 x 1 1/2	237 1/2	69.0	3020	353.3	1766	106.0	.001103
"	" x 1 1/4	213 1/2	62.0	2586	313.5	1567	88.0	.001136
"	" x 1	189 1/2	55.0	2177	272.2	1361	70.0	.001172
"	" x 7/8	178 1/2	51.5	1983	251.8	1259	61.5	.001190
"	" x 3/4	166	48.0	1794	231.5	1157	52.5	.001210
"	" x 5/8	154	44.5	1611	211.3	1056	44.0	.001230
"	" x 1/2	142	41.0	1434	191.2	956	35.0	.001250
"	" x 3/8	130	37.5	1263	171.2	856	26.0	.001271

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2 1/2 per cent. over this must be allowed. See page 7.

Let  $\delta$  = deflection,  $K$  = deflection coefficient, and  $L$  = span in feet, then  $\delta = K \times L^4$ .

For full explanations of tables, see notes commencing page 108.

For formulae, explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND GIRDERS.

Safe Distributed Loads, in Tons.

Reference Mark.	Size, D x B inches.	SPANS IN FEET.													
		8	10	12	14	16	18	20	22	24	26	28	30	32	34
132B	16 x 14	Rivets 3-in. diameter.													
130B	15 1/2 x "														
128B	15 x "								71.6	65.6	60.6	56.2	52.5	49.2	46.3
126B	14 1/2 x "						77.8	70.1	63.7	58.4	53.9	50.0	46.7	43.8	
124B	14 x "				87.3	76.4	67.9	61.1	55.6	50.9	47.0	43.7	40.7	38.2	
123B	13 3/4 x "			94.7	81.2	71.0	63.1	56.8	51.6	47.3	43.7	40.6	37.9		
122B	13 1/2 x "		105	87.6	75.1	65.7	58.4	52.5	47.8	43.8	40.4	37.5	35.0		
121B	13 1/4 x "	121	96.7	80.5	69.0	60.4	53.7	48.3	43.9	40.3	37.2	34.5	32.2		
120B	13 x "	110	88.2	73.5	63.0	55.2	49.0	44.1	40.1	36.8	33.9	31.5	29.4		
119B	12 1/2 x "	100	79.9	66.6	57.1	50.0	44.4	40.0	36.3	33.3	30.8	28.6			
108B	15 x 14	Rivets 3-in. diameter.													
106B	14 1/2 x "														
104B	14 x "				81.9	71.7	63.7	57.4	52.1	47.8	44.1	41.0	38.2	35.8	
103B	13 3/2 x "		88.3	75.7	66.2	58.9	53.0	48.1	44.1	40.8	37.8	35.3			
102B	13 1/2 x "		97.3	81.1	69.5	60.8	54.1	48.6	44.2	40.5	37.4	34.8	32.4		
101B	13 1/4 x "		88.7	73.9	63.4	55.4	49.3	44.4	40.3	36.9	34.1	31.7	29.6		
100B	13 x "	100	80.1	66.8	57.3	50.1	44.5	40.1	36.4	33.4	30.8	28.6	26.7		
99B	12 3/4 x "	89.6	71.6	59.7	51.2	44.8	39.8	35.8	32.6	29.8	27.6	25.6			
88B	15 x 12	Rivets 3-in. diameter.													
86B	14 1/2 x "														
84B	14 x "		75.0	64.3	56.3	50.0	45.0	40.9	37.5	34.6	32.2	30.0	28.1		
83B	13 3/2 x "		68.3	59.0	51.6	45.9	41.3	37.5	34.4	31.8	29.5	27.5			
82B	13 1/2 x "	94.0	75.2	62.7	53.7	47.0	41.8	37.6	34.2	31.3	28.9	26.8	25.1		
81B	13 1/4 x "	84.9	67.8	56.5	48.5	42.4	37.7	34.0	30.8	28.3	26.1	24.2	22.6		
80B	13 x "	75.8	60.6	50.5	43.3	37.9	33.7	30.3	27.5	25.2	23.2	21.6	20.2		
79B	12 3/4 x "	66.7	53.3	44.4	38.1	33.3	29.6	26.7	24.2	22.2	20.5	19.0			

Tabular loads to right of full zigzag line produce deflection greater than 1/26th of an inch per foot of span.  
 Girders supporting tabular loads to left of dotted zigzag line require stiffeners to prevent web buckling.  
 Girders supporting tabular loads printed in ordinary type have rivets at 6 inches pitch.  
 Girders supporting tabular loads printed in italics require a closer pitch of rivets. See page 55.  
 Safe working stress = 7.5 tons per square inch, equal to a factor of safety of 4. Ends of girders simply supported.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND GIRDERS.

Composition and Properties.



Composed of		Weight per foot in lbs.	Area in square inches.	Maxi- mum Moment of Inertia. X-X	Maxi- mum Modulus of Section. X-X	Safe Distributed Load on 1 foot Span for		Deflection Coefficient. X-X
Two Steel Joists.	Plates, each Flange to form.					Girder.	1 in. Plate Width.	
12 x 6a	14 x 2	301	87.7	3106	388.2	1941	123.0	.001172
"	" x 1 1/2	277	80.7	2722	351.2	1756	107.0	.001210
"	" x 1 1/4	253 1/2	73.7	2362	315.0	1575	91.5	.001250
"	" x 1 1/2	229 1/2	66.7	2031	280.2	1401	76.0	.001293
"	" x 1	205 1/2	59.7	1711	244.5	1222	60.5	.001339
"	" x 7/8	194	56.2	1562	227.8	1136	52.5	.001364
"	" x 3/4	182	52.7	1419	210.2	1051	45.0	.001389
"	" x 5/8	170	49.2	1281	193.3	966	37.5	.001415
"	" x 1/2	158	45.7	1147	176.5	882	30.0	.001442
"	" x 3/8	146	42.2	1019	159.9	799	22.5	.001471
12 x 6b	14 x 1 1/2	233 1/2	67.8	2257	300.9	1504	91.5	.001250
"	" x 1 1/4	209 1/2	60.8	1926	265.7	1328	76.0	.001293
"	" x 1	185 1/2	53.8	1606	229.4	1147	60.5	.001339
"	" x 7/8	174	50.3	1457	211.9	1059	52.5	.001364
"	" x 3/4	162	46.8	1314	194.6	973	45.0	.001389
"	" x 5/8	150	43.3	1175	177.4	887	37.5	.001415
"	" x 1/2	138	39.8	1042	160.3	801	30.0	.001442
"	" x 3/8	126	36.3	914	143.3	716	22.5	.001471
12 x 5	12 x 1 1/2	189	54.8	1806	240.8	1204	91.5	.001250
"	" x 1 1/4	168 1/2	48.8	1529	210.9	1054	76.0	.001293
"	" x 1	148	42.8	1260	180.1	900	60.5	.001339
"	" x 7/8	138	39.8	1136	165.2	826	52.5	.001364
"	" x 3/4	127 1/2	36.8	1015	150.4	752	45.0	.001389
"	" x 5/8	117 1/2	33.8	899	135.7	678	37.5	.001415
"	" x 1/2	107 1/2	30.8	787	121.2	606	30.0	.001442
"	" x 3/8	97	27.8	680	106.7	533	22.5	.001471

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2 1/2 per cent. over this must be allowed. See page 7.

Let  $\delta$  = deflection,  $K$  = deflection coefficient, and  $L$  = span in feet, then  $\delta = K \times LA$ .

For full explanations of tables, see notes commencing page 108.

For formulae, explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND GIRDERS.

Safe Distributed Loads, in Tons.

Reference Mark.	Size, D x B inches.	SPANS IN FEET.												
		6	8	10	12	14	16	18	20	22	24	26	28	30
68B	13 x 14									56.2	51.5	47.5	44.1	41.2
66B	12½ x "								60.2	54.2	49.3	45.2	41.7	38.7
64B	12 x "								66.9	58.5	52.0	46.8	42.5	39.0
63B	11½ x "								71.9	61.7	55.3	48.0	43.2	39.2
62B	11 x "								79.1	65.9	56.5	49.5	44.9	39.6
61B	11½ x "								90.0	72.0	60.0	51.4	45.0	40.0
60B	11 x "								81.2	64.9	54.1	46.4	40.6	36.1
59B	10½ x "								96.0	72.4	57.9	48.3	41.4	36.2
48B	13 x 12									49.5	45.0	41.2	38.1	35.3
46B	12½ x "								53.8	47.8	43.0	39.1	35.9	33.1
44B	12 x "								61.2	52.5	45.9	40.8	36.7	33.4
43B	11½ x "								56.0	48.0	42.0	37.3	33.6	30.5
42B	11 x "								61.0	50.9	43.6	38.2	33.9	30.5
41B	11½ x "								68.7	55.0	45.8	39.3	34.3	30.5
40B	11 x "								81.5	61.1	48.9	40.8	34.9	30.6
39B	10½ x "								71.5	53.6	42.9	35.7	30.6	26.8
28B	11 x 14									43.0	39.4	36.4		
26B	10½ x "								45.6	41.0	37.3	34.2		
24B	10 x "								49.9	43.7	38.8	34.9	31.7	29.1
23B	9½ x "								53.2	45.6	39.9	35.5	31.9	29.0
22B	9 x "								58.0	48.4	41.5	36.3	32.2	29.0
21B	9½ x "								65.3	52.2	43.5	37.3	32.6	29.0
20B	9 x "								77.6	58.2	46.5	38.8	33.2	29.1
19B	8½ x "								68.3	51.2	40.9	34.1	29.3	25.6
14B	10 x 12								47.5	41.1	35.9	31.9	28.7	26.1
13B	9½ x "								52.5	43.7	37.5	32.8	29.2	26.3
12B	9 x "								47.5	39.6	34.0	29.7	26.4	23.8
11B	9½ x "								53.3	42.6	35.5	30.5	26.7	23.1
10B	9 x "								63.1	47.3	37.8	31.5	27.0	23.6
9B	8½ x "								55.2	41.4	33.1	27.6	23.6	20.7

Tabular loads to right of full zigzag line produce deflection greater than 1/26th of an inch per foot of span  
 Girders supporting tabular loads to left of dotted zigzag line require stiffeners to prevent web buckling  
 Girders supporting tabular loads printed in *italics* have rivets at 5 inches pitch  
 Girders supporting tabular loads printed in *italics* require a closer pitch of rivets. See page 55  
 Safe working stress = 7.5 tons per square inch, equal to a factor of safety of 4. Ends of girders simply supported

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND GIRDERS.

### Composition and Properties.



Composed of		Weight per foot in lbs.	Area in square inches.	Maxi- mum Moment of Inertia. x-x	Maxi- mum Modulus of Section. x-x	Safe Distributed Load on 1 foot Span for		Deflection Coefficient. x-x
Two Steel Joists.	Plates, each Flange to form.					Girder.	1 in. Plate Width.	
10 x 6	14 x 1 1/2	229 1/2	66.7	1607	247.3	1236	76.5	.001442
"	" x 1 1/4	205 1/2	59.7	1356	216.9	1084	63.5	.001500
"	" x 1	181 1/2	52.7	1124	187.3	936	50.5	.001563
"	" x 7/8	170	49.2	1015	172.7	863	44.0	.001596
"	" x 3/4	158	45.7	910	158.3	791	37.5	.001630
"	" x 5/8	146	42.2	810	144.0	720	31.5	.001667
"	" x 1/2	134	38.7	714	129.9	649	25.0	.001704
"	" x 3/8	122	35.2	623	115.9	579	18.5	.001744
10 x 5	12 x 1 1/2	185	53.6	1286	197.9	989	76.5	.001442
"	" x 1 1/4	164 1/2	47.6	1076	172.1	860	63.5	.001500
"	" x 1	144	41.6	881	146.8	734	50.5	.001563
"	" x 7/8	134	38.6	790	134.4	672	44.0	.001596
"	" x 3/4	123 1/2	35.6	702	122.1	610	37.5	.001630
"	" x 5/8	113 1/2	32.6	618	109.9	549	31.5	.001667
"	" x 1/2	103 1/2	29.6	538	97.8	489	25.0	.001704
"	" x 3/8	93	26.6	461	85.8	429	18.5	.001744
8 x 6	14 x 1 1/2	215 1/2	62.5	1040	189.1	945	62.0	.001704
"	" x 1 1/4	191 1/2	55.5	861	164.0	820	51.0	.001785
"	" x 1	167 1/2	48.5	698	139.7	698	40.5	.001875
"	" x 7/8	156	45.0	623	127.8	639	35.5	.001923
"	" x 3/4	144	41.5	551	116.1	580	30.0	.001974
"	" x 5/8	132	38.0	483	104.5	522	25.0	.002027
"	" x 1/2	120	34.5	419	93.1	465	20.0	.002083
"	" x 3/8	108	31.0	358	81.8	409	15.0	.002143
8 x 5	12 x 1 1/2	140	40.4	575	115.0	575	40.5	.001875
"	" x 1 1/4	130	37.4	512	105.0	525	35.5	.001923
"	" x 1	119 1/2	34.4	452	95.1	475	30.0	.001974
"	" x 7/8	109 1/2	31.4	395	85.3	426	25.0	.002027
"	" x 3/4	99 1/2	28.4	341	75.7	378	20.0	.002083
"	" x 5/8	89	25.4	289	66.2	331	15.0	.002143

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2 1/2 per cent. over this must be allowed. See page 7.

Let  $\delta$  = deflection,  $K$  = deflection coefficient, and  $L$  = span in feet, then  $\delta = K \times L^4$ .

For full explanations of tables, see notes commencing page 108.

For formulae, explanations of properties, etc., see Part IV.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND GIRDERS.

Safe Distributed Loads, in Tons.

Reference Mark.	Size, D x B inches.	SPANS IN FEET.													
		14	16	18	20	22	24	26	28	30	32	34	36	40	44
206C	20 x 24									297	278	263	248	228	203
204C	28½ x "	Rivets ¾-in. diameter.								296	277	259	244	230	207
292C	28 x "							321	296	275	257	240	226	214	192
290C	27½ x "				354	322	295	273	253	236	221	208	197	177	161
288C	27 x "			361	325	295	270	250	232	216	203	191	180	162	147
286C	26½ x "		369	328	295	268	246	227	211	197	185	173	164	147	134
284C	26 x "	379	332	295	266	241	221	204	190	177	166	156	147	133	121
283C	25½ x "	359	314	279	251	228	209	193	179	167	157	148	139	125	114
282C	25½ x "	338	295	263	236	215	197	182	169	158	148	139	131	118	107
281C	25¼ x "	317	277	246	222	202	185	171	158	148	139	130	123	111	101
280C	25 x "	296	259	230	207	188	173	159	148	138	130	122	115	104	94.0
276C	25 x 24	Rivets ¾-in. diameter.								240	225	212	200	180	163
274C	24½ x "								238	223	209	196	185	167	152
272C	24 x "						257	237	220	206	193	181	171	154	140
270C	23½ x "				283	257	236	218	202	189	177	167	157	142	129
268C	23 x "			287	258	235	215	199	184	172	161	152	143	129	117
266C	22½ x "	333	292	259	233	212	194	180	167	156	146	137	130	117	106
264C	22 x "	298	261	232	209	190	174	161	149	139	131	123	116	104	94.9
263C	21½ x "	281	246	218	197	179	164	151	141	131	123	116	109	98.3	89.4
262C	21½ x "	264	231	205	184	168	154	142	132	123	115	108	102	92.3	83.9
261C	21¼ x "	246	216	192	172	157	144	133	123	115	108	101	95.8	86.3	78.4
260C	21 x "	229	201	178	160	146	134	123	115	107	100	94.4	89.2	80.2	73.0

Tabular loads to right of full zigzag line produce deflection greater than 1/26th of an inch per foot of span.  
Girders supporting tabular loads to left of dotted zigzag line require stiffeners to prevent web buckling.  
Girders supporting tabular loads printed in ordinary type have rivets at 6 inches pitch.  
Girders supporting tabular loads printed in italics require a closer pitch of rivets. See page 56  
Safe working stress = 7.5 tons per square inch, equal to a factor of safety of 4. Ends of girders simply supported.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND GIRDERS.

### Composition and Properties.



Composed of		Weight per foot in lbs.	Area in square inches.	Maxi- mum Moment of Inertia. X-X	Maxi- mum Modulus of Section. X-X	Safe Distributed Load on 1 foot Span for		Deflection Coefficient. X-X
Three Steel Joists.	Plates, each Flange to form.					Girder.	1 in. Plate Width.	
24 x 7½	24 x 2½	713½	208.1	25848	1782.7	8913	303.5	.000646
"	" x 2½	672½	196.1	23658	1660.2	8301	272.5	.000658
"	" x 2	632	184.1	21545	1538.9	7694	242.0	.000669
"	" x 1½	591	172.1	19505	1418.5	7092	211.0	.000681
"	" x 1½	550	160.1	17539	1299.0	6495	180.5	.000694
"	" x 1½	509½	148.1	15643	1180.7	5903	150.5	.000707
"	" x 1	469½	136.1	13818	1062.9	5314	120.0	.000721
"	" x ¾	448	130.1	12932	1004.5	5022	105.0	.000728
"	" x ¾	428	124.1	12062	946.1	4730	90.0	.000735
"	" x ¾	407½	118.1	11210	887.9	4439	75.0	.000742
"	" x ½	387	112.1	10374	829.9	4149	60.0	.000750
20 x 7½	24 x 2½	680½	198.4	17979	1438.3	7191	254.0	.000750
"	" x 2½	639½	186.4	16356	1335.2	6676	228.0	.000765
"	" x 2	599	174.4	14799	1233.2	6166	202.0	.000781
"	" x 1½	558	162.4	13305	1132.3	5661	176.5	.000797
"	" x 1½	517½	150.4	11873	1032.4	5162	151.0	.000815
"	" x 1½	476½	138.4	10502	933.5	4667	125.5	.000833
"	" x 1	435½	126.4	9191	835.6	4178	100.0	.000852
"	" x ¾	416½	120.4	8557	786.9	3934	87.5	.000862
"	" x ¾	395	114.4	7938	738.4	3692	75.0	.000872
"	" x ¾	374½	108.4	7333	690.2	3451	62.5	.000880
"	" x ½	354	102.4	6742	642.1	3210	50.0	.000893

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

Let  $\delta$  = deflection,  $K$  = deflection coefficient, and  $L$  = span in feet, then  $\delta = K \times L^4$ .

For full explanations of tables, see notes commencing page 108.

For formulae, explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND GIRDERS.

Safe Distributed Loads, in Tons.

Reference Mark.	Size, D x B inches.	SPANS IN FEET.													
		14	16	18	20	22	24	26	28	30	32	34	36	40	44
256C	23 x 24										194	182	172	155	141
254C	22½ x "									205	191	179	168	159	143
252C	22 x "									188	175	164	155	146	131
250C	21½ x "									171	160	150	141	133	120
248C	21 x "									155	145	136	128	121	109
246C	20½ x "									139	130	122	114	108	97
244C	20 x "									123	115	108	101	95	86
243C	19½ x "									108	101	95	89	83	77
242C	19½ x "									100	94	88	83	77	73
241C	19½ x "									93	87	82	77	72	68
240C	19 x "									85	80	75	71	66	63
										78	73	68	64	59	55
232C	20 x 21									133	125	117	111	99	90
230C	19½ x "									121	113	107	101	90	82
228C	19 x "									109	102	96	90	81	74
226C	18½ x "									97	91	85	81	73	66
224C	18 x "									85	80	75	71	64	58
223C	17½ x "									79	75	70	66	60	55
222C	17½ x "									74	69	65	61	55	50
221C	17½ x "									68	64	60	57	51	46
220C	17 x "									62	59	55	52	47	42
										56	53	50	47	42	38
212C	19 x 21									124	116	109	103	93	85
210C	18½ x "									112	105	99	93	84	76
208C	18 x "									101	95	89	84	76	69
206C	17½ x "									90	84	79	75	67	61
204C	17 x "									85	79	74	70	63	57
203C	16½ x "									79	74	69	65	58	52
202C	16½ x "									73	68	64	60	53	47
201C	16½ x "									67	63	59	55	48	42
200C	16 x "									62	58	54	51	44	38

Tabular loads to right of full zigzag line produce deflection greater than 1/26th of an inch per foot of span.

Girders supporting tabular loads to left of dotted zigzag line require stiffeners to prevent web buckling.

Girders supporting tabular loads printed in ordinary type have rivets at 6 inches pitch.

Girders supporting tabular loads printed in italics require a closer pitch of rivets. See page 57.

Safe working stress = 7½ tons per square inch, equal to a factor of safety of 4. Ends of girders simply supported.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND GIRDERS.

Composition and Properties.



Composed of		Weight per foot in lbs.	Area in square inches.	Maxi- mum Moment of Inertia. X-X	Maxi- mum Modulus of Section. X-X	Safe Distributed Load on 1 foot Span for		Deflection Coefficient. X-X
Three Steel Joists.	Plates, each Flange to form.					Girder.	1 in. Plate Width.	
18 x 7	24 x 2 $\frac{1}{2}$	638 $\frac{1}{2}$	186.1	14259	1239.9	6199	229.5	.000815
"	" x 2 $\frac{1}{2}$	598	174.1	12888	1145.7	5728	206.0	.000833
"	" x 2	557	162.1	11577	1052.5	5262	182.5	.000852
"	" x 1 $\frac{3}{4}$	516 $\frac{1}{2}$	150.1	10324	960.4	4802	159.0	.000872
"	" x 1 $\frac{1}{2}$	475 $\frac{1}{2}$	138.1	9128	869.4	4347	136.0	.000893
"	" x 1 $\frac{1}{4}$	434 $\frac{1}{2}$	126.1	7988	779.3	3896	113.0	.000914
"	" x 1	394	114.1	6902	690.2	3451	90.0	.000937
"	" x $\frac{7}{8}$	373 $\frac{1}{2}$	108.1	6379	645.9	3229	79.0	.000949
"	" x $\frac{3}{4}$	353	102.1	5869	601.9	3009	67.5	.000961
"	" x $\frac{5}{8}$	332 $\frac{1}{2}$	96.1	5371	558.1	2790	56.0	.000974
"	" x $\frac{1}{2}$	312 $\frac{1}{2}$	90.1	4887	514.5	2572	45.0	.000987
16 x 6	21 x 2	475	138.6	7983	798.3	3991	162.5	.000937
"	" x 1 $\frac{3}{4}$	439 $\frac{1}{2}$	128.1	7078	726.0	3630	141.5	.000961
"	" x 1 $\frac{1}{2}$	403 $\frac{1}{2}$	117.6	6218	654.6	3273	121.0	.000987
"	" x 1 $\frac{1}{4}$	368	107.1	5402	584.1	2920	100.5	.001013
"	" x 1	332 $\frac{1}{2}$	96.6	4630	514.4	2572	80.5	.001041
"	" x $\frac{7}{8}$	314 $\frac{1}{2}$	91.4	4259	479.9	2399	70.0	.001056
"	" x $\frac{3}{4}$	296 $\frac{1}{2}$	86.1	3899	445.6	2228	60.0	.001071
"	" x $\frac{5}{8}$	279	80.9	3548	411.4	2057	50.0	.001087
"	" x $\frac{1}{2}$	261	75.6	3208	377.4	1887	40.0	.001103
15 x 6	21 x 2	466	136.0	7066	743.8	3719	152.5	.000987
"	" x 1 $\frac{3}{4}$	430 $\frac{1}{2}$	125.5	6250	675.6	3378	133.0	.001013
"	" x 1 $\frac{1}{2}$	395	115.0	5477	608.7	3043	113.5	.001041
"	" x 1 $\frac{1}{4}$	359	104.5	4746	542.4	2712	94.5	.001071
"	" x 1	323 $\frac{1}{2}$	94.0	4056	477.2	2386	75.5	.001103
"	" x $\frac{7}{8}$	305 $\frac{1}{2}$	88.7	3725	444.8	2224	66.0	.001119
"	" x $\frac{3}{4}$	287 $\frac{1}{2}$	83.5	3405	412.7	2063	56.5	.001136
"	" x $\frac{5}{8}$	270	78.2	3094	380.8	1904	47.0	.001151
"	" x $\frac{1}{2}$	252	73.0	2792	349.0	1745	37.5	.001172

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2 $\frac{1}{2}$  per cent. over this must be allowed. See page 7.

Let  $\delta$  = deflection, K = deflection coefficient, and L = span in feet, then  $\delta = K \times L^4$ .

For full explanations of tables see notes commencing page 108.

For formulae, explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND GIRDERS.

Safe Distributed Loads, in Tons.

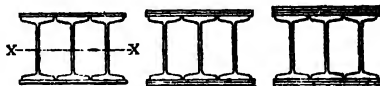
Reference Mark.	Size, D x B inches.	SPANS IN FEET.															
		10	12	14	16	18	20	22	24	26	28	30	32	34	36		
188C	18 x 18	Rivets 3-in. diameter.															
186C	17½ x "																
184C	17 x "	153	131	114	102	91	83	76	70	65	61	57	53	50	49		
183C	16½ x "	170	141	121	106	94	84	77	70	65	60	56	53	49	47		
182C	16½ x "	156	130	111	97	86	77	70	64	59	55	51	48	45	43		
181C	16½ x "	143	118	101	88	79	71	64	59	54	50	47	44	41	39		
180C	16 x "	129	107	91	80	71	64	58	53	49	45	42	40	37	35		
179C	15½ x "	115	95	82	72	63	57	52	48	44	41	38	36	33	31		
172C	18 x 20	Rivets 3-in. diameter.															
170C	17½ x "																
168C	17 x "	135	112	104	96	88	80	75	70	65	61	57	53	50	49		
166C	16½ x "	151	134	120	109	100	92	86	81	75	70	65	61	57	53		
164C	16 x "	152	133	118	106	96	88	81	75	70	65	61	57	53	50		
163C	15½ x "	198	165	141	124	110	99	90	82	76	70	65	61	57	53		
162C	15½ x "	184	153	131	115	102	92	83	76	70	65	61	57	53	50		
161C	15½ x "	170	142	121	106	94	85	77	70	65	61	57	53	50	48		
160C	15 x "	156	130	111	97	86	77	71	64	59	55	51	48	45	43		
159C	14½ x "	142	118	102	88	79	71	64	59	55	51	48	45	43	41		
148C	17 x 20	Rivets 3-in. diameter.															
146C	16½ x "																
144C	16 x "	141	123	109	98	89	82	75	70	65	61	57	53	50	49		
143C	15½ x "	152	130	114	101	91	83	76	70	65	61	57	53	50	49		
142C	15½ x "	168	140	120	105	93	84	76	70	65	61	57	53	50	49		
141C	15½ x "	154	128	110	96	85	77	70	64	59	55	51	48	45	43		
140C	15 x "	140	116	99	87	77	69	63	58	53	49	46	43	41	39		
139C	14½ x "	126	105	89	78	69	62	57	52	48	44	41	39	37	35		

Tabular loads to right of full zigzag line produce deflection greater than 1/26th of an inch per foot of span.  
Girders supporting tabular loads to left of dotted zigzag line require stiffeners to prevent web buckling.  
Girders supporting tabular loads printed in ordinary type have rivets at 6 inches pitch.  
Girders supporting tabular loads printed in italics require a closer pitch of rivets. See page 57  
Safe working stress = 7½ tons per square inch, equal to a factor of safety of 4. Ends of girders simply supported.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND GIRDERS.

Composition and Properties.



Composed of		Weight per foot in lbs.	Area in square inches.	Maxi- mum Moment of Inertia. X-X	Maxi- mum Modulus of Section. X-X	Safe Distributed Load on 1 foot Span for		Deflection Coefficient. X-X
Three Steel Joists.	Plates, each Flange to form.					Girder.	1 in. Plate Width.	
15 x 5	18 x 1½	313	91.0	4309	478.8	2394	113.5	.001041
"	" x 1½	282½	82.0	3696	422.4	2112	94.5	.001071
"	" x 1	252	73.0	3117	366.7	1833	75.5	.001103
"	" x ¾	236½	68.5	2840	339.1	1695	66.0	.001119
"	" x ¾	221½	64.0	2571	311.7	1558	56.5	.001136
"	" x ¾	206	59.5	2311	284.4	1422	47.0	.001151
"	" x ¾	191	55.0	2058	257.2	1286	37.5	.001172
"	" x ¾	175½	50.5	1813	230.2	1161	28.0	.001190
14 x 6a	20 x 2	446½	130.3	5938	659.8	3299	143.0	.001041
"	" x 1½	412½	120.3	5246	599.6	2998	124.5	.001071
"	" x 1½	378½	110.3	4593	540.3	2701	106.0	.001103
"	" x 1½	344½	100.3	3977	482.1	2410	88.0	.001136
"	" x 1	310½	90.3	3397	424.7	2123	70.5	.001172
"	" x ¾	293½	85.3	3121	396.3	1981	61.5	.001190
"	" x ¾	276½	80.3	2853	368.1	1840	52.5	.001210
"	" x ¾	259½	75.3	2593	340.1	1700	44.0	.001230
"	" x ¾	242½	70.3	2342	312.3	1561	35.0	.001250
"	" x ¾	225½	65.3	2099	284.6	1423	26.0	.001271
14 x 6b	20 x 1½	345½	100.5	4349	511.7	2558	106.0	.001103
"	" x 1½	311½	90.5	3733	452.5	2262	88.0	.001136
"	" x 1	277½	80.5	3154	394.2	1971	70.5	.001172
"	" x ¾	260½	75.5	2877	365.3	1826	61.5	.001190
"	" x ¾	243½	70.5	2609	336.6	1683	52.5	.001210
"	" x ¾	226½	65.5	2349	308.1	1540	44.0	.001230
"	" x ¾	209½	60.5	2098	279.8	1399	35.0	.001250
"	" x ¾	192½	55.5	1856	251.6	1258	26.0	.001271

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

Let  $\delta$  = deflection,  $K$  = deflection coefficient, and  $L$  = span in feet, then  $\delta = K \times L^4$ .

For full explanations of tables, see notes commencing page 108.

For formulae, explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND GIRDERS.

Safe Distributed Loads, in Tons.

Reference Mark.	Size, D x B inches.	SPANS IN FEET.															
		8	10	12	14	16	18	20	22	24	26	28	30	32	34		
132C	16 x 20												99	69	92	98	132
130C	15 1/2 x "												90	28	84	27	90
128C	15 x "								113	103	94	87	81	176	70	96	8
126C	14 1/2 x "								101	92	84	77	72	367	5	63	3
124C	14 x "				148	127	111	98	87	80	73	68	263	3	59	1	55
123C	13 3/4 x "				165	138	118	103	91	82	75	168	8	63	5	59	0
122C	13 1/2 x "				153	128	109	97	78	51	76	60	66	58	9	54	7
121C	13 1/4 x "				176	141	118	101	88	2	78	4	70	6	64	2	58
120C	13 x "				163	129	108	92	4	80	9	71	8	64	7	58	8
119C	12 3/4 x "				147	118	98	0	84	0	73	5	65	4	58	8	53
108C	15 x 20												98	4	90	2	33
106C	14 1/2 x "												87	1	79	0	73
104C	14 x "								107	95	87	1	79	0	73	7	68
103C	13 3/4 x "								92	83	0	75	4	69	2	63	9
102C	13 1/2 x "				128	110	96	1	85	4	76	4	69	8	64	0	59
101C	13 1/4 x "				141	118	101	88	4	78	6	70	7	64	2	58	9
100C	13 x "				162	129	108	92	3	80	8	71	8	64	6	58	7
99C	12 3/4 x "				147	117	97	7	83	7	73	2	65	1	58	6	53
88C	15 x 18												90	3	82	0	75
86C	14 1/2 x "												69	4	64	5	60
84C	14 x "												67	5	61	3	56
83C	13 3/4 x "												61	9	56	3	51
82C	13 1/2 x "												51	6	47	7	44
81C	13 1/4 x "												43	4	40	3	37
80C	13 x "												43	4	39	1	36
79C	12 3/4 x "												34	9	32	4	30

Tabular loads to right of full zigzag line produce deflection greater than 1/26 of an inch per foot of span.

Girders supporting tabular loads to left of dotted zigzag line require stiffeners to prevent web buckling.

Girders supporting tabular loads printed in ordinary type have rivets at 6 inches pitch.

Girders supporting tabular loads printed in italics require a closer pitch of rivets. See page 58.

Safe working stress = 7.5 tons per square inch, equal to a factor of safety of 4. Ends of girders simply supported.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND GIRDERS.

Composition and Properties.



Composed of		Weight per foot in lbs.	Area in square inches.	Maxi- mum Moment of Inertia. X-X	Maxi- mum Modulus of Section. X-X	Safe Distributed Load on 1 foot Span for		Deflection Coefficient. X-X
Three Steel Joists.	Plates, each Flange to form.					Girder.	1 in. Plate Width.	
12 x 6a	20 x 2	437½	127·6	446	557·6	2788	123·0	·001172
"	" x 1½	403½	117·6	391	505·4	2527	107·0	·001210
"	" x 1¼	369½	107·6	3406	454·1	2270	91·5	·001250
"	" x 1½	335½	97·6	2937	405·1	2025	76·0	·001293
"	" x 1	301½	87·6	2483	354·7	1773	60·5	·001339
"	" x ¾	284½	82·6	2271	330·4	1652	52·5	·001364
"	" x ¾	267½	77·6	2067	306·3	1531	45·0	·001389
"	" x ¾	250½	72·6	1871	282·4	1412	37·5	·001415
"	" x ¾	233½	67·6	1682	258·8	1294	30·0	·001442
"	" x ¾	216½	62·6	1500	235·3	1176	22·5	·001471
12 x 6b	20 x 1½	339½	98·8	3248	433·1	2165	91·5	·001250
"	" x 1¼	305½	88·8	2779	383·3	1916	76·0	·001293
"	" x 1	271½	78·8	2324	332·1	1660	60·5	·001339
"	" x ¾	254½	73·8	2113	307·4	1537	52·5	·001364
"	" x ¾	237½	68·8	1909	282·8	1414	45·0	·001389
"	" x ¾	220½	63·8	1713	258·6	1293	37·5	·001415
"	" x ¾	203½	58·8	1524	234·4	1172	30·0	·001442
"	" x ¾	186½	53·8	1342	210·5	1052	22·5	·001471
12 x 5	18 x 1½	283	82·2	2709	361·2	1806	91·5	·001250
"	" x 1¼	252½	73·2	2293	316·3	1581	76·0	·001293
"	" x 1	222	64·2	1891	270·1	1350	60·5	·001339
"	" x ¾	206½	59·7	1703	247·8	1239	52·5	·001364
"	" x ¾	191½	55·2	1523	225·6	1128	45·0	·001389
"	" x ¾	176	50·7	1349	203·6	1018	37·5	·001415
"	" x ¾	161	46·2	1181	181·7	908	30·0	·001442
"	" x ¾	145½	41·7	1020	160·0	800	22·5	·001471

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

Let  $\delta$  = deflection,  $K$  = deflection coefficient, and  $L$  = span in feet, then  $\delta = K \times L^2$ .

For full explanations of tables, see notes commencing page 108.

For formulae, explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND GIRDERS.

Safe Distributed Loads, in Tons.

Reference Mark.	Size, D x B inches.	SPANS IN FEET.												
		6	8	10	12	14	16	18	20	22	24	26	28	30
68C	13 x 20								88.9	80.8	74.1	68.4	63.5	59.3
66C	12½ x "							86.9	78.8	71.1	65.1	60.1	55.8	
64C	12 x "							84.6	75.2	67.7	61.5	56.4	52.1	48.3
63C	11½ x "							82.2	72.3	64.6	58.8	53.1	48.1	
62C	11¼ x "							80.8	70.8	63.8	57.5	52.2	47.9	44.2
61C	11 x "							78.4	68.5	61.5	55.2	49.9	45.3	41.3
60C	11 x "							76.0	66.1	59.1	52.8	47.5	42.9	38.5
59C	10½ x "							73.6	63.7	56.7	50.4	45.1	40.5	36.1
48C	13 x 18								74.2	67.5	61.9	57.1	52.3	49.5
46C	12½ x "							80.7	71.7	64.5	58.6	53.8	49.6	46.1
44C	12 x "							78.3	69.3	62.1	56.2	51.4	47.2	43.3
43C	11½ x "							75.9	66.9	59.7	53.8	48.9	44.7	41.3
42C	11¼ x "							73.5	64.5	57.3	51.4	46.5	42.3	38.9
41C	11 x "							71.1	62.1	54.9	49.0	44.1	39.9	36.5
40C	11 x "							68.7	59.7	52.5	46.6	41.7	37.5	34.1
39C	10½ x "							66.3	57.3	50.1	44.2	39.3	35.1	31.7
28C	11 x 20										61.6	56.5	52.1	
25C	10½ x "										59.2	54.1	49.7	
24C	10 x "										56.8	51.7	47.3	
23C	9½ x "										54.4	49.3	44.9	
22C	9¼ x "										52.0	46.9	42.5	
21C	9 x "										49.6	44.5	40.1	
20C	9 x "										47.2	42.1	37.7	
19C	8½ x "										44.8	39.7	35.3	
14C	10 x 18													
13C	9½ x "													
12C	9¼ x "													
11C	9 x "													
10C	9 x "													
9C	8½ x "													

Tabular loads to right of full zigzag line produce deflection greater than 1/26th of an inch per foot of span  
 Girders supporting tabular loads to left of dotted zigzag line require stiffeners to prevent web buckling.  
 Girders supporting tabular loads printed in ordinary type have rivets at 6 inches pitch  
 Girders supporting tabular loads printed in italics require a closer pitch of rivets. See page 58  
 Safe working stress = 7.5 tons per square inch, equal to a factor of safety of 4. Ends of girders simply supported

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND GIRDERS.

Composition and Properties.



Composed of		Weight per foot in lbs.	Area in square inches.	Maxi- mum Moment of Inertia. X-X	Maxi- mum Modulus of Section. X-X	Safe Distributed Load on 1 foot Span for		Deflection Coefficient. X-X
Three Steel Joists.	Plates, each Flange to form.					Girder.	1 in. Plate Width.	
10 x 6	20 x 1 1/2	333 1/2	97.0	2311	355.6	1778	76.5	.001442
"	" x 1 1/4	299 1/2	87.0	1954	312.7	1563	63.5	.001500
"	" x 1	265 1/2	77.0	1625	270.8	1354	50.5	.001563
"	" x 7/8	248 1/2	72.0	1470	250.3	1251	44.0	.001596
"	" x 3/4	231 1/2	67.0	1322	229.9	1149	37.5	.001630
"	" x 5/8	214 1/2	62.0	1180	209.7	1048	31.5	.001667
"	" x 1/2	197 1/2	57.0	1044	189.8	949	25.0	.001704
"	" x 3/8	180 1/2	52.0	914	170.1	850	18.5	.001744
10 x 5	18 x 1 1/2	277	80.4	1930	296.9	1484	76.5	.001442
"	" x 1 1/4	246 1/2	71.4	1614	258.2	1291	63.5	.001500
"	" x 1	216	62.4	1322	220.3	1101	50.5	.001563
"	" x 7/8	200 1/2	57.9	1184	201.6	1008	44.0	.001596
"	" x 3/4	185 1/2	53.4	1053	183.1	915	37.5	.001630
"	" x 5/8	170	48.9	927	164.8	824	31.5	.001667
"	" x 1/2	155	44.4	807	146.7	733	25.0	.001704
"	" x 3/8	139 1/2	39.9	692	128.7	643	18.5	.001744
8 x 6	20 x 1 1/2	312 1/2	90.8	1491	271.1	1355	62.0	.001704
"	" x 1 1/4	278 1/2	80.8	1238	235.7	1178	51.0	.001785
"	" x 1	244 1/2	70.8	1007	201.4	1007	40.5	.001875
"	" x 7/8	227 1/2	65.8	900	184.6	923	35.5	.001923
"	" x 3/4	210 1/2	60.8	798	168.0	840	30.0	.001974
"	" x 5/8	193 1/2	55.8	702	151.7	758	25.0	.002027
"	" x 1/2	176 1/2	50.8	610	135.6	678	20.0	.002083
"	" x 3/8	159 1/2	45.8	524	119.8	599	15.0	.002143
8 x 5	18 x 1	210	60.7	862	172.5	862	40.5	.001875
"	" x 7/8	195	56.2	767	157.5	787	35.5	.001923
"	" x 3/4	179 1/2	51.7	677	142.6	713	30.0	.001974
"	" x 5/8	164	47.2	592	128.0	640	25.0	.002027
"	" x 1/2	149	42.7	511	113.5	577	20.0	.002083
"	" x 3/8	133 1/2	38.2	434	99.3	496	15.0	.002143

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2 1/2 per cent. over this must be allowed. See page 7.

Let  $\delta$  = deflection,  $K$  = deflection coefficient, and  $L$  = span in feet, then  $\delta = K \times L^3$ .

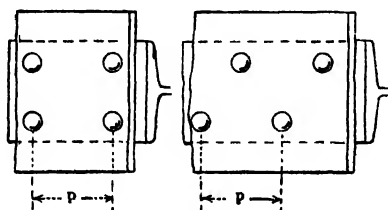
For full explanations of tables, see notes commencing page 108.

For formulae, explanations of properties, &c., see Part IV.



# COMPOUND GIRDERS.

Minimum Spans in Feet for various Rivet Pitches.



Straight Pitch.

Staggered or Reeled Pitch.

P = 3", 4", or 6".

The tables on this and the two following pages give the nearest pitch of rivets in *even* inches which should be adopted if the girders, pages 20 to 29, are used to support the full safe distributed tabular loads in italics.

*Example*.—Required rivet pitch for girder 274A, page 20, to support tabular load of 98 tons on a span of 32 feet.

*Answer*.—See girder 274A in this table, which gives 4 in. as the required pitch, the minimum span being 29.1 feet.

Reference Mark.	Size, D x B inches.	Minimum Spans in Feet for Rivet Pitches of		
		3-in.	4-in.	6-in.
296A	29 x 12	24.4	32.6	48.8
294A	28½ x "	21.8	29.1	43.6
292A	28 x "	19.2	25.6	38.4
290A	27½ x "	16.6	22.1	33.1
288A	27 x "	13.9	18.5	27.8
286A	26½ x "	11.2	14.9	22.4
284A	26 x "	8.6	11.4	17.1
283A	25½ x "	7.3	9.7	14.5
282A	25½ x "	6.0	7.9	11.9
281A	25½ x "	4.7	6.3	9.4
280A	25 x "	3.6	4.6	6.9

Rivets ½-in. diam.

276A	25 x 12	24.4	32.6	48.8
274A	24½ x "	21.8	29.1	43.6
272A	24 x "	19.3	25.7	38.5
270A	23½ x "	16.4	21.9	32.8
268A	23 x "	14.0	18.7	28.0
266A	22½ x "	11.4	15.2	22.7
264A	22 x "	8.7	11.6	17.4
263A	21½ x "	7.4	9.9	14.8
262A	21½ x "	6.1	8.1	12.2
261A	21½ x "	4.8	6.4	9.6
260A	21 x "	3.6	4.8	7.1

Rivets ¼-in. diam.

For safe loads and properties of these girders, see pages 20 and 21.

For full explanations of tables, see notes commencing page 108.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND GIRDERS.

Minimum Spans in Feet for  
various Rivet Pitches.

Reference Mark.	Size, D x B inches.	Minimum Spans in Feet for Rivet Pitches of			Reference Mark.	Size, D x B inches.	Minimum Spans in Feet for Rivet Pitches of		
		3-in.	4-in.	6-in.			3-in.	4-in.	6-in.
256A	23 x 12	24.7	32.9	49.3	188A	18 x 9	15.2	20.2	30.3
254A	22½ x "	22.1	29.5	44.2	186A	17½ x "	12.5	16.6	25.0
252A	22 x "	19.8	26.4	39.6	184A	17 x "	9.7	12.9	19.4
250A	21½ x "	17.0	22.6	33.9	183A	16½ x "	8.4	11.2	16.7
248A	21 x "	14.3	19.1	28.6	182A	16½ x "	7.0	9.3	14.0
246A	20½ x "	11.7	15.6	23.4	181A	16½ x "	5.6	7.5	11.2
244A	20 x "	9.0	12.0	18.0	180A	16 x "	4.5	5.9	8.9
243A	19½ x "	7.7	10.3	15.4	179A	15½ x "	2.9	3.9	5.8
242A	19½ x "	6.4	8.5	12.7	Rivets ½-in. diam.				
241A	19½ x "	5.1	6.7	10.1	172A	18 x 10	21.9	29.2	43.8
240A	19 x "	3.8	5.0	7.5	170A	17½ x "	19.8	26.3	39.5
Rivets ½-in. diam.					168A	17 x "	16.2	21.6	32.4
232A	20 x 10	22.1	29.4	44.2	166A	16½ x "	13.2	17.6	26.4
230A	19½ x "	19.2	25.6	38.3	164A	16 x "	10.3	13.7	20.5
228A	19 x "	16.2	21.6	32.4	163A	15½ x "	8.7	11.6	17.3
226A	18½ x "	13.3	17.7	26.5	162A	15½ x "	7.3	9.7	14.6
224A	18 x "	10.3	13.7	20.5	161A	15½ x "	5.8	7.7	11.6
223A	17½ x "	8.8	11.7	17.5	160A	15 x "	4.4	5.8	8.7
222A	17½ x "	7.3	9.7	14.5	159A	14½ x "	3.0	3.9	5.9
221A	17½ x "	5.8	7.8	11.6	Rivets ½-in. diam.				
220A	17 x "	4.3	5.8	8.6	148A	17 x 10	16.7	22.2	33.3
Rivets ½-in. diam.					146A	16½ x "	13.7	18.3	27.4
212A	19 x 10	22.0	29.3	44.0	144A	16 x "	10.7	14.3	21.4
210A	18½ x "	19.1	25.5	38.3	143A	15½ x "	9.2	12.2	18.3
208A	18 x "	16.2	21.6	32.4	142A	15½ x "	7.7	10.2	15.3
206A	17½ x "	13.2	17.6	26.4	141A	15½ x "	6.2	8.2	12.3
204A	17 x "	10.2	13.6	20.4	140A	15 x "	4.7	6.2	9.3
203A	16½ x "	8.8	11.7	17.5	139A	14½ x "	3.2	4.3	6.4
202A	16½ x "	7.3	9.7	14.5	Rivets ½-in. diam.				
201A	16½ x "	5.8	7.7	11.5					
200A	16 x "	4.3	5.8	8.6					
Rivets ½-in. diam.									

For safe loads and properties of these girders, see pages 22 to 25.

For full explanations of tables, see notes commencing page 108.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND GIRDERS.

Minimum Spans in Feet for  
various Rivet Pitches.

Reference Mark.	Size, D x B inches.	Minimum Spans in Feet for Rivet Pitches of			Reference Mark.	Size, D x B inches.	Minimum Spans in Feet for Rivet Pitches of		
		3-in.	4-in.	6-in.			3-in.	4-in.	6-in.
132A	16 x 10	21·7	28·9	43·4	68A	13 x 10	16·3	21·8	32·6
130A	15½ x "	18·9	25·2	37·8	66A	12½ x "	13·4	17·9	26·8
128A	15 x "	16·1	21·4	32·1	64A	12 x "	10·4	13·9	20·8
126A	14½ x "	13·2	17·5	26·3	63A	11¾ x "	9·1	12·1	18·1
124A	14 x "	10·2	13·6	20·5	62A	11½ x "	7·6	10·1	15·2
123A	13¾ x "	8·8	11·7	17·5	61A	11¼ x "	6·1	8·2	12·2
122A	13½ x "	7·3	9·7	14·6	60A	11 x "	4·6	6·2	9·2
121A	13¼ x "	5·8	7·8	11·6	59A	10¾ x "	3·2	4·3	6·4
120A	13 x "	4·4	5·8	8·7	48A	13 x 9	15·1	20·2	30·2
119A	12¾ x "	3·0	4·0	5·9	46A	12½ x "	12·6	16·7	25·1
Rivets ½-in. diam.					44A	12 x "	9·9	13·2	19·8
108A	15 x 10	16·5	22·0	33·0	43A	11¾ x "	8·6	11·4	17·2
106A	14½ x "	13·6	18·1	27·2	42A	11½ x "	7·2	9·6	14·4
104A	14 x "	10·6	14·1	21·2	41A	11¼ x "	5·9	7·8	11·7
103A	13¾ x "	9·1	12·2	18·2	40A	11 x "	4·5	6·0	9·0
102A	13½ x "	7·6	10·2	15·2	39A	10¾ x "	3·2	4·2	6·3
101A	13¼ x "	6·1	8·2	12·2	28A	11 x 10	16·3	21·7	32·6
100A	13 x "	4·7	6·2	9·3	26A	10½ x "	13·6	18·1	27·1
99A	12¾ x "	3·2	4·3	6·4	24A	10 x "	10·7	14·	21·4
Rivets ½-in. diam.					23A	9¾ x "	9·3	12·	18·5
88A	15 x 9	15·3	20·4	30·6	22A	9½ x "	7·8	10·4	15·6
86A	14½ x "	12·7	16·9	25·3	21A	9¼ x "	6·4	8·5	12·7
84A	14 x "	10·0	13·3	19·9	20A	9 x "	4·9	6·5	9·7
83A	13¾ x "	8·6	11·5	17·2	19A	8¾ x "	3·4	4·5	6·8
82A	13½ x "	7·3	9·7	14·5	14A	10 x 9	9·8	13·0	19·6
81A	13¼ x "	5·9	7·8	11·7	13A	9¾ x "	8·5	11·3	17·0
80A	13 x "	4·5	6·0	9·0	12A	9½ x "	7·2	9·6	14·3
79A	12¾ x "	3·1	4·2	6·2	11A	9¼ x "	5·8	7·8	11·7
Rivets ½-in. diam.					10A	9 x "	4·5	6·0	9·0
					9A	8¾ x "	3·2	4·2	6·3
					Rivets ½-in. diam.				

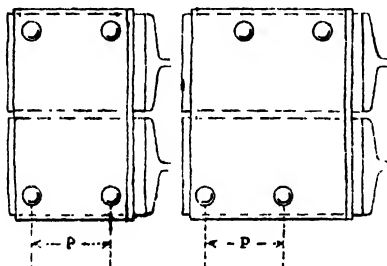
For safe loads and properties of these girders, see pages 26 to 29.

For full explanations of tables, see notes commencing page 108.



## COMPOUND GIRDERS.

Minimum Spans in Feet for  
various Rivet Pitches.



Straight  
Pitch.

Staggered or Reeled  
Pitch.

$P = 3", 4", \text{ or } 6".$

The tables on this and the two following pages give the nearest pitch of rivets in *even* inches which should be adopted if the girders, pages 30 to 39, are used to support the full safe distributed tabular loads in *italics*.

**Example:**—Required rivet pitch for girder 288B, page 30, to support tabular load of 240·6 tons on a span of 18 feet.

**Answer:**—See girder 288B in this table, which gives 3 ins. as the required pitch, the minimum span being 16·5 feet.

Reference Mark.	Size, D x B inches.	Minimum Spans in Feet for Rivet Pitches of		
		3-in.	4-in.	6-in.
296B	29 x 16	30·1	40·1	60·2
294B	28½ x "	26·7	35·6	53·4
292B	28 x "	23·3	31·1	46·6
290B	27½ x "	19·9	26·5	39·8
288B	27 x "	16·5	22·0	33·0
286B	26½ x "	13·1	17·5	26·2
284B	26 x "	9·8	13·1	19·6
283B	25½ x "	8·2	10·9	16·4
282B	25¼ x "	6·6	8·8	13·2
281B	25¼ x "	5·1	6·8	10·2
280B	25 x "	3·7	4·9	7·3

Rivets ½-in. diam.

276B	25 x 16	30·2	40·2	60·3
274B	24½ x "	26·9	35·8	53·7
272B	24 x "	23·5	31·3	47·0
270B	23½ x "	19·9	26·5	39·8
268B	23 x "	16·7	22·3	33·4
266B	22½ x "	13·4	17·8	26·7
264B	22 x "	10·1	13·4	20·1
263B	21½ x "	8·4	11·2	16·8
262B	21¼ x "	6·8	9·1	13·6
261B	21¼ x "	5·3	7·1	10·6
260B	21 x "	3·8	5·1	7·6

Rivets ¾-in. diam.

For safe loads and properties of these girders, see pages 30 and 31.

For full explanations of tables, see notes commencing page 108.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND GIRDERS.

Minimum Spans in Feet for various Rivet Pitches.

Reference Mark.	Size, D x B inches.	Minimum Spans in Feet for Rivet Pitches of			Reference Mark.	Size, D x B inches.	Minimum Spans in Feet for Rivet Pitches of		
		3-in.	4-in.	6-in.			3-in.	4-in.	6-in.
256B	23 x 16	30·8	41·1	61·6	188B	18 x 12	18·7	24·9	37·4
254B	22½ x "	27·5	36·6	54·9	186B	17½ x "	15·2	20·3	30·4
252B	22 x "	24·1	32·1	48·2	184B	17 x "	11·7	15·6	23·3
250B	21½ x "	20·7	27·6	41·4	183B	16¾ x "	9·9	13·2	19·8
248B	21 x "	17·3	23·1	34·6	182B	16½ x "	8·2	10·9	16·3
246B	20½ x "	13·9	18·6	27·8	181B	16¼ x "	6·5	8·6	12·9
244B	20 x "	10·5	14·0	21·0	180B	16 x "	4·8	6·4	9·5
243B	19¾ x "	8·9	11·8	17·7	179B	15¾ x "	3·2	4·2	6·3
242B	19½ x "	7·3	9·7	14·5	Rivets ½-in. diam.				
241B	19¼ x "	5·7	7·5	11·3	172B	18 x 14	28·8	38·4	57·6
240B	19 x "	4·1	5·5	8·2	170B	17½ x "	24·9	33·1	49·7
Rivets ½-in. diam.					168B	17 x "	20·9	27·9	41·8
232B	20 x 14	29·0	38·6	57·9	166B	16½ x "	16·9	22·5	33·7
230B	19½ x "	25·0	33·3	49·9	164B	16 x "	12·9	17·2	25·7
228B	19 x "	20·9	27·9	41·9	163B	15¾ x "	10·9	14·5	21·8
226B	18½ x "	16·9	22·5	33·8	162B	15½ x "	8·9	11·9	17·8
224B	18 x "	12·9	17·1	25·7	161B	15¼ x "	7·0	9·3	14·0
223B	17¾ x "	10·9	14·5	21·7	160B	15 x "	5·1	6·8	10·2
222B	17½ x "	8·9	11·9	17·8	159B	14¾ x "	3·4	4·5	6·7
221B	17¼ x "	7·0	9·3	13·9	Rivets ½-in. diam.				
220B	17 x "	5·1	6·8	10·2	148B	17 x 14	21·8	29·0	43·5
Rivets ½-in. diam.					146B	16½ x "	17·7	23·6	35·4
212B	19 x 14	28·8	38·5	57·6	144B	16 x "	13·6	18·2	27·2
210B	18½ x "	24·8	33·1	49·7	143B	15¾ x "	11·6	15·5	23·2
208B	18 x "	20·9	27·8	41·8	142B	15½ x "	9·6	12·7	19·1
206B	17½ x "	16·8	22·5	33·7	141B	15¼ x "	7·6	10·1	15·1
204B	17 x "	12·8	17·1	25·6	140B	15 x "	5·6	7·5	11·2
203B	16¾ x "	10·8	14·5	21·6	139B	14¾ x "	3·8	5·0	7·5
202B	16½ x "	8·9	11·8	17·7	Rivets ½-in. diam.				
201B	16¼ x "	7·0	9·3	13·9					
200B	16 x "	5·1	6·8	10·2					
Rivets ½-in. diam.									

For safe loads and properties of these girders, see pages 82 to 85.

For full explanations of tables, see notes commencing page 108.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND GIRDERS.

Minimum Spans in Feet for  
various Rivet Pitches.

Refer- ence Mark.	Size, D x B inches.	Minimum Spans in Feet for Rivet Pitches of			Refer- ence Mark.	Size, D x B inches.	Minimum Spans in Feet for Rivet Pitches of		
		3-in.	4-in.	6-in.			3-in.	4-in.	6-in.
132B	16 x 14	28.6	38.2	57.2	68B	13 x 14	21.3	28.4	42.6
130B	15½ x "	24.7	32.9	49.3	66B	12½ x "	17.4	23.2	34.8
128B	15 x "	20.8	27.7	41.5	64B	12 x "	13.5	18.0	26.9
126B	14½ x "	16.8	22.5	33.7	63B	11½ x "	11.5	15.3	22.9
124B	14 x "	12.9	17.1	25.7	62B	11¼ x "	9.5	12.7	19.0
123B	13¾ x "	10.9	14.5	21.8	61B	11¼ x "	7.5	10.0	15.0
122B	13½ x "	8.9	11.9	17.8	60B	11 x "	5.6	7.5	11.2
121B	13¼ x "	7.0	9.4	14.0	59B	10¾ x "	3.8	5.0	7.5
120B	13 x "	5.1	6.9	10.3	48B	13 x 12	18.9	25.3	37.9
119B	12¾ x "	3.4	4.5	6.7	46B	12½ x "	15.6	20.7	31.1
Rivets ¾-in. diam.					44B	12 x "	12.1	16.1	24.2
108B	15 x 14	21.6	28.8	43.1	43B	11½ x "	10.4	13.8	20.7
106B	14½ x "	17.6	23.5	35.2	42B	11¼ x "	8.6	11.5	17.2
104B	14 x "	13.5	18.1	27.1	41B	11¼ x "	6.9	9.2	13.8
103B	13¾ x "	11.5	15.4	23.0	40B	11 x "	5.2	6.9	10.3
102B	13½ x "	9.5	12.7	19.0	39B	10¾ x "	3.5	4.7	7.0
101B	13¼ x "	7.6	10.0	15.1	28B	11 x 14	21.6	28.8	43.2
100B	13 x "	5.6	7.5	11.2	26B	10½ x "	17.8	23.7	35.5
99B	12¾ x "	3.7	5.0	7.4	24B	10 x "	13.9	18.5	27.7
Rivets ¾-in. diam.					23B	9¾ x "	11.9	15.9	23.8
88B	15 x 12	19.1	25.5	38.2	22B	9¼ x "	9.9	13.2	19.8
86B	14½ x "	15.7	20.9	31.3	21B	9¼ x "	8.0	10.6	16.9
84B	14 x "	12.1	16.2	24.3	20B	9 x "	6.0	8.0	12.0
83B	13¾ x "	10.4	13.8	20.8	19B	8¾ x "	4.1	5.4	8.1
82B	13½ x "	8.6	11.5	17.2	14B	10 x 12	12.0	16.0	23.9
81B	13¼ x "	6.9	9.2	13.7	13B	9¾ x "	10.3	13.7	20.5
80B	13 x "	5.2	6.9	10.3	12B	9¼ x "	8.6	11.4	17.1
79B	12¾ x "	3.5	4.6	6.9	11B	9¼ x "	6.9	9.1	13.7
Rivets ¾-in. diam.					10B	9 x "	5.2	6.9	10.3
					9B	8¾ x "	3.5	4.7	7.0
					Rivets ¾-in. diam.				

For safe loads and properties of these girders, see pages 36 to 39.

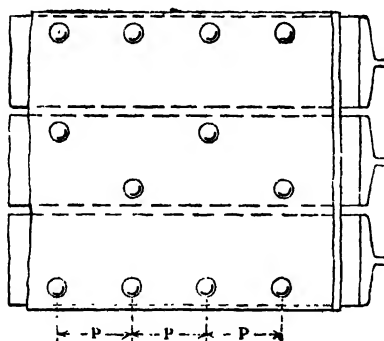
For full explanations of tables, see notes commencing page 108.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND GIRDERS.

Minimum Spans in Feet for  
various Rivet Pitches.



$P = 3", 4", \text{ or } 6".$

The tables on this and the two following pages give the nearest pitch of rivets in even inches, spaced as shown on sketch, which should be adopted if the girders, pages 40 to 49, are used to support the full safe distributed tabular loads in italics.

*Example*.—Required rivet pitch for girder 264C, page 40, to support tabular load of 298 tons on a span of 14 feet.

*Answer*.—See girder 264C in this table, which gives 4 ins. as the required pitch, the minimum span being 13·4 feet.

Reference Mark.	Size, D x B inches.	Minimum Spans in Feet for Rivet Pitches of		
		3-in.	4-in.	6-in.
296C	29 x 24	30·1	40·1	60·2
294C	28½ x "	26·7	35·6	53·4
292C	28 x "	23·3	31·1	46·6
290C	27½ x "	19·9	26·5	39·7
288C	27 x "	16·5	22·0	32·9
286C	26½ x "	13·1	17·5	26·2
284C	26 x "	9·8	13·1	19·6
283C	25½ x "	8·2	10·9	16·4
282C	25½ x "	6·6	8·8	13·2
281C	25½ x "	5·1	6·8	10·2
280C	25 x "	3·7	4·9	7·3

Rivets ¾-in. diam.

276C	25 x 24	30·2	40·2	60·4
274C	24½ x "	26·8	35·8	53·6
272C	24 x "	23·5	31·3	46·9
270C	23½ x "	20·1	26·8	40·2
268C	23 x "	16·7	22·3	33·4
266C	22½ x "	13·2	17·6	26·4
264C	22 x "	10·1	13·4	20·1
263C	21½ x "	8·4	11·2	16·8
262C	21½ x "	6·8	9·1	13·6
261C	21½ x "	5·3	7·0	10·5
260C	21 x "	4·0	5·3	7·9

Rivets ¾-in. diam.

For safe loads and properties of these girders, see pages 40 and 41.

For full explanations of tables, see notes commencing page 108.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND GIRDERS.

Minimum Spans in Feet for  
various Rivet Pitches.

Refer- ence Mark.	Size, D x B inches.	Minimum Spans in Feet for Rivet Pitches of			Refer- ence Mark.	Size, D x B inches.	Minimum Spans in Feet for Rivet Pitches of		
		3-in.	4-in.	6-in.			3-in.	4-in.	6-in.
256C	23 x 24	30.8	41.1	61.6	188C	18 x 18	18.7	25.0	37.4
254C	22½ x "	26.5	35.2	52.9	186C	17½ x "	15.2	20.3	30.4
252C	22 x "	24.1	32.1	48.2	184C	17 x "	11.7	15.6	23.3
250C	21½ x "	20.7	27.6	41.4	183C	16½ x "	9.9	13.2	19.8
248C	21 x "	17.3	23.1	34.6	182C	16¼ x "	8.2	10.9	16.3
246C	20½ x "	13.9	18.5	27.8	181C	16¼ x "	6.4	8.6	12.9
244C	20 x "	10.5	14.0	21.0	180C	16 x "	4.8	6.4	9.5
243C	19½ x "	8.9	11.8	17.7	179C	15½ x "	3.2	4.2	6.3
242C	19¼ x "	7.3	9.7	14.5	Rivets ¾-in. diam.				
241C	19¼ x "	5.6	7.5	11.2	172C	18 x 20	27.1	36.2	54.2
240C	19 x "	4.1	5.5	8.2	170C	17½ x "	23.4	31.2	46.8
Rivets ¾-in. diam.					168C	17 x "	19.6	26.1	39.2
232C	20 x 21	28.9	38.5	57.7	166C	16½ x "	15.8	21.1	31.6
230C	19½ x "	25.0	33.3	49.9	164C	16 x "	12.9	17.2	25.9
228C	19 x "	20.9	27.9	41.8	163C	15½ x "	10.2	13.6	20.3
226C	18½ x "	16.9	22.5	33.7	162C	15¼ x "	8.3	11.1	16.6
224C	18 x "	12.9	17.1	25.7	161C	15¼ x "	6.5	8.6	13.0
223C	17½ x "	10.9	14.5	21.7	160C	15 x "	4.7	6.3	9.5
222C	17¼ x "	8.9	11.9	17.8	159C	14½ x "	3.1	4.1	6.1
221C	17¼ x "	7.0	9.3	13.9	Rivets ¾-in. diam.				
220C	17 x "	5.6	7.5	11.1	148C	17 x 20	20.5	27.4	41.0
Rivets ¾-in. diam.					146C	16½ x "	16.7	22.2	33.8
212C	19 x 21	28.9	38.5	57.7	144C	16 x "	13.4	17.9	26.8
210C	18½ x "	24.9	33.2	49.8	143C	15½ x "	10.9	14.5	21.7
208C	18 x "	20.9	27.8	41.7	142C	15¼ x "	8.9	11.9	17.8
206C	17½ x "	16.9	22.5	33.7	141C	15¼ x "	7.1	9.4	14.1
204C	17 x "	12.8	17.1	25.6	140C	15 x "	5.2	7.0	10.4
203C	16½ x "	10.8	14.4	21.7	139C	14½ x "	3.5	4.6	6.9
202C	16¼ x "	8.9	11.9	17.7	Rivets ¾-in. diam.				
201C	16¼ x "	7.0	9.3	13.9					
200C	16 x "	5.6	7.5	11.1					

For safe loads and properties of these girders, see pages 42 to 45.

For full explanations of tables, see notes commencing page 108.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND GIRDERS.

Minimum Spans in Feet for  
various Rivet Pitches.

Reference Mark.	Size, D x B inches.	Minimum Spans in Feet for Rivet Pitches of			Reference Mark.	Size, D x B inches.	Minimum Spans in Feet for Rivet Pitches of		
		3-in.	4-in.	6-in.			3-in.	4-in.	6-in.
132C	16 x 20	26-9	35-9	53-8	68C	13 x 20	20-1	26-8	40-2
130C	15½ x "	23-3	31-0	46-5	66C	12½ x "	16-4	21-8	32-8
128C	15 x "	19-5	26-0	39-0	64C	12 x "	12-6	16-8	25-2
126C	14½ x "	15-8	21-1	31-6	63C	11½ x "	10-7	14-3	21-5
124C	14 x "	12-0	16-0	24-0	62C	11 x "	8-9	11-8	17-7
123C	13½ x "	10-2	13-6	20-3	61C	11¼ x "	7-0	9-4	14-0
122C	13¼ x "	8-3	11-1	16-6	60C	11 x "	5-2	6-9	10-4
121C	13¼ x "	6-5	8-7	13-0	59C	10½ x "	3-5	4-6	6-9
120C	13 x "	4-8	6-4	9-5	48C	13 x 18	19-0	25-3	37-9
119C	12¾ x "	3-1	4-1	6-2	46C	12½ x "	15-6	20-7	31-1
Rivets ¾-in. diam.					44C	12 x "	12-1	16-1	24-2
108C	15 x 20	20-3	27-1	40-6	43C	11½ x "	10-4	13-8	20-7
106C	14½ x "	16-6	22-1	33-1	42C	11¼ x "	8-6	11-5	17-2
104C	14 x "	12-7	17-0	25-4	41C	11 x "	6-9	9-2	13-8
103C	13½ x "	10-8	14-4	21-6	40C	11 x "	5-2	6-9	10-3
102C	13¼ x "	8-9	11-9	17-8	39C	10½ x "	3-5	4-7	7-0
101C	13¼ x "	7-0	9-3	14-0	28C	11 x 20	20-4	27-2	40-7
100C	13 x "	5-2	6-9	10-4	26C	10½ x "	16-8	22-4	33-5
99C	12¾ x "	3-5	4-6	6-9	24C	10 x "	13-1	17-4	26-1
Rivets ¾-in. diam.					23C	9½ x "	11-2	14-9	22-4
88C	15 x 18	19-1	25-5	38-2	22C	9½ x "	9-3	12-4	18-6
86C	14½ x "	15-7	20-9	31-4	21C	9¼ x "	7-4	9-9	14-8
84C	14 x "	12-2	16-2	24-3	20C	9 x "	5-6	7-4	11-1
83C	13½ x "	10-4	13-8	20-8	19C	8½ x "	3-8	5-0	7-5
82C	13¼ x "	8-6	11-5	17-2	14C	10 x 18	12-0	16-0	23-9
81C	13¼ x "	6-9	9-2	13-7	13C	9½ x "	10-3	13-7	20-5
80C	13 x "	5-2	6-9	10-3	12C	9½ x "	8-6	11-4	17-1
79C	12¾ x "	3-5	4-6	6-9	11C	9¼ x "	6-9	9-1	13-7
Rivets ¾-in. diam.					10C	9 x "	5-2	6-9	10-3
					9C	8½ x "	3-5	4-6	7-0
					Rivets ¾-in. diam.				

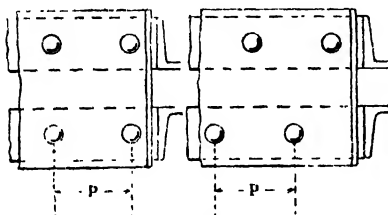
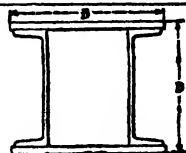
For safe loads and properties of these girders, see pages 46 to 49.

For full explanations of tables, see notes commencing page 108.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND GIRDERS.

Minimum Spans in Feet for  
various Rivet Pitches.



Straight  
Pitch.

Staggered or Reeled  
Pitch.

P = 3", 4", or 6".

The tables on this page give the nearest pitch of rivets in *even* inches which should be adopted if the girders, pages 74-75, are used to support the full safe distributed tabular loads in italics.

*Example*.—Required rivet pitch for girder 15E, page 74, to support tabular load of 51.7 tons on a span of 10 feet.

*Answer*.—See girder 15E in this table, which gives 4 ins. as the required pitch, the minimum span being 9.9 feet.

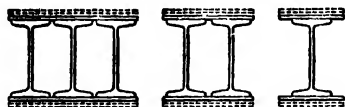
Reference Mark.	Size, D x B inches.	Minimum Spans in Feet for Rivet Pitches of		
		3-in.	4-in.	6-in.
29E	16½ x 16	12.5	16.7	25.0
28E	16½ x "	10.1	13.4	20.1
27E	16 x "	7.7	10.2	15.3
26E	16½ x 12	8.7	11.5	17.3
25E	16½ x "	6.9	9.2	13.8
24E	16 x "	5.2	6.9	10.3
23E	13½ x 16	13.7	18.2	27.3
22E	13½ x "	10.5	14.0	21.0
21E	13 x "	8.1	10.7	16.1
20E	13½ x 12	7.3	9.7	14.5
19E	13 x "	5.5	7.4	11.0
18E	11½ x 16	13.1	17.5	26.2
17E	11½ x "	10.7	14.2	21.3
16E	11 x "	8.2	11.0	16.4
15E	11½ x 12	7.4	9.9	14.8
14E	11 x "	5.7	7.5	11.3
13E	10½ x 16	13.2	17.6	26.3
12E	10½ x "	10.8	14.3	21.5
11E	10 x "	8.3	11.1	16.6
10E	10½ x 12	7.5	10.0	15.0
9E	10 x "	5.8	7.7	11.5
8E	9½ x 16	10.8	14.5	21.7
7E	9 x "	8.4	11.2	16.8
6E	9½ x 12	7.6	10.1	15.2
5E	9 x "	5.9	7.8	11.7
4E	8½ x 16	10.9	14.6	21.8
3E	8 x "	8.5	11.4	17.0
2E	8½ x 12	7.7	10.2	15.4
1E	8 x "	5.9	7.9	11.8

Rivets ½-in. diam.

For safe loads and properties of these girders, see pages 74 and 75.

For full explanations of tables, see notes commencing page 108.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND GIRDERS.

Arranged in Descending Order of Carrying Capacity.

Composed of			Weight per foot in lbs.	Maximum Moment of Resistance, in foot tons.	Composed of			Weight per foot in lbs.	Maximum Moment of Resistance, in foot tons.
1	Steel Joist(s).	Plates, each Flange to form.			1	Steel Joist(s).	Plates, each Flange to form.		
3	24 x 7 $\frac{1}{2}$	24 x 2 $\frac{1}{2}$	713 $\frac{1}{2}$	1114.2	3	24 x 7 $\frac{1}{2}$	24 x $\frac{5}{8}$	407 $\frac{1}{2}$	554.9
3	24 x 7 $\frac{1}{2}$	24 x 2 $\frac{1}{2}$	672 $\frac{1}{2}$	1037.6	3	18 x 7	24 x 1 $\frac{1}{2}$	475 $\frac{1}{2}$	543.4
3	24 x 7 $\frac{1}{2}$	24 x 2	632	961.8	2	24 x 7 $\frac{1}{2}$	16 x 1 $\frac{1}{2}$	367	541.3
3	20 x 7 $\frac{1}{2}$	24 x 2 $\frac{1}{2}$	680 $\frac{1}{2}$	898.9	1	24 x 7 $\frac{1}{2}$	12 x 2 $\frac{1}{2}$	308	523.1
3	24 x 7 $\frac{1}{2}$	24 x 1 $\frac{1}{2}$	591	886.6	3	20 x 7 $\frac{1}{2}$	24 x 1	435 $\frac{1}{2}$	522.2
3	20 x 7 $\frac{1}{2}$	24 x 2 $\frac{1}{2}$	639 $\frac{1}{2}$	834.5	3	24 x 7 $\frac{1}{2}$	24 x $\frac{1}{2}$	387	518.7
3	24 x 7 $\frac{1}{2}$	24 x 1 $\frac{1}{2}$	550	811.9	2	18 x 7	16 x 2 $\frac{1}{2}$	426	516.6
3	18 x 7	24 x 2 $\frac{1}{2}$	638 $\frac{1}{2}$	774.9	2	20 x 7 $\frac{1}{2}$	16 x 2	399 $\frac{1}{2}$	513.8
3	20 x 7 $\frac{1}{2}$	24 x 2	599	770.8	3	16 x 6	21 x 2	475	498.9
2	24 x 7 $\frac{1}{2}$	16 x 2 $\frac{1}{2}$	475 $\frac{1}{2}$	742.6	2	24 x 7 $\frac{1}{2}$	16 x 1 $\frac{1}{2}$	339 $\frac{1}{2}$	491.9
3	24 x 7 $\frac{1}{2}$	24 x 1 $\frac{1}{2}$	509 $\frac{1}{2}$	737.9	3	20 x 7 $\frac{1}{2}$	24 x $\frac{1}{2}$	415 $\frac{1}{2}$	491.8
3	18 x 7	24 x 2 $\frac{1}{2}$	598	716.1	3	18 x 7	24 x 1 $\frac{1}{2}$	434 $\frac{1}{2}$	487.1
3	20 x 7 $\frac{1}{2}$	24 x 1 $\frac{1}{2}$	558	707.6	1	24 x 7 $\frac{1}{2}$	12 x 2 $\frac{1}{2}$	287 $\frac{1}{2}$	482.2
2	24 x 7 $\frac{1}{2}$	16 x 2 $\frac{1}{2}$	448 $\frac{1}{2}$	691.4	2	18 x 7	16 x 2 $\frac{1}{2}$	398 $\frac{1}{2}$	477.3
3	24 x 7 $\frac{1}{2}$	24 x 1	468 $\frac{1}{2}$	664.3	2	20 x 7 $\frac{1}{2}$	16 x 1 $\frac{1}{2}$	372	471.7
3	18 x 7	24 x 2	557	657.8	3	15 x 6	21 x 2	466	464.9
3	20 x 7 $\frac{1}{2}$	24 x 1 $\frac{1}{2}$	517 $\frac{1}{2}$	645.3	3	20 x 7 $\frac{1}{2}$	24 x 0 $\frac{3}{4}$	395	461.5
2	24 x 7 $\frac{1}{2}$	16 x 2	421 $\frac{1}{2}$	641.2	3	16 x 6	21 x 1 $\frac{1}{2}$	439 $\frac{1}{2}$	453.7
3	24 x 7 $\frac{1}{2}$	24 x $\frac{1}{2}$	448	627.8	2	24 x 7 $\frac{1}{2}$	16 x 1	312 $\frac{1}{2}$	442.8
3	18 x 7	24 x 1 $\frac{1}{2}$	516 $\frac{1}{2}$	600.2	1	24 x 7 $\frac{1}{2}$	12 x 2	267	441.5
2	20 x 7 $\frac{1}{2}$	16 x 2 $\frac{1}{2}$	453 $\frac{1}{2}$	599.3	2	18 x 7	16 x 2	371 $\frac{1}{2}$	438.5
3	24 x 7 $\frac{1}{2}$	24 x $\frac{5}{8}$	428	591.3	3	20 x 7 $\frac{1}{2}$	24 x $\frac{5}{8}$	374 $\frac{1}{2}$	431.4
2	24 x 7 $\frac{1}{2}$	16 x 1 $\frac{1}{2}$	394	591.0	3	18 x 7	24 x 1	394	431.4
3	20 x 7 $\frac{1}{2}$	24 x 1 $\frac{1}{2}$	476 $\frac{1}{2}$	583.4	2	20 x 7 $\frac{1}{2}$	16 x 1 $\frac{1}{2}$	345	430.1
2	20 x 7 $\frac{1}{2}$	16 x 2 $\frac{1}{2}$	426 $\frac{1}{2}$	556.3	1	20 x 7 $\frac{1}{2}$	12 x 2 $\frac{1}{2}$	297	426.7

In this table the single, double, and triple joist-compound girders are brought together and arranged in descending order of strength. The method of selection of a suitable girder for any system of loading is as follows:—

Calculate Maximum Bending Moment in foot tons. See Part IV.

Refer to column headed "Maximum Moment of Resistance, foot tons."

Any girder of which the Maximum Moment of Resistance is not less than the calculated Bending Moment will have sufficient carrying capacity.

Safe working stress = 7.5 tons per square inch.

Note deflection, web-buckling, and rivet-pitch limitations.

For safe loads and properties of these girders, see pages 20 to 49.

For full explanations of tables, see notes commencing page 108.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND GIRDERS.

Arranged in Descending Order of Carrying Capacity.



Composed of			Weight per foot in lbs.	Maximum Moment of Resistance in foot tons.	Composed of			Weight per foot in lbs.	Maximum Moment of Resistance in foot tons.
1 2 3	Steel Joist(s).	Plates, each Flange to form.			1 2 3	Steel Joist(s).	Plates, each Flange to form.		
3	15 x 6	21 x 1 1/2	430 1/2	422.3	3	15 x 6	21 x 1 1/2	369	339.0
2	24 x 7 1/2	16 x 1 1/2	299	418.5	3	14 x 6a	20 x 1 1/2	378 1/2	337.7
3	14 x 6a	20 x 2	446 1/2	412.4	2	16 x 6	14 x 2	317	332.6
3	16 x 6	21 x 1 1/2	403 1/2	409.1	2	20 x 7 1/2	16 x 1 1/2	277	327.8
3	18 x 7	24 x 1 1/2	373 1/2	403.7	2	18 x 7	16 x 1 1/2	280	324.6
3	20 x 7 1/2	24 x 1 1/2	354	401.3	1	20 x 7 1/2	12 x 1 1/2	235 1/2	324.1
1	24 x 7 1/2	12 x 1 1/2	246 1/2	401.2	3	18 x 7	24 x 1 1/2	312 1/2	321.6
2	18 x 7	16 x 1 1/2	344 1/2	400.1	3	16 x 6	21 x 1	332 1/2	321.5
2	24 x 7 1/2	16 x 1 1/2	285 1/2	394.2	1	24 x 7 1/2	12 x 1 1/2	206	321.2
1	20 x 7 1/2	12 x 2 1/2	276 1/2	392.2	3	14 x 6b	20 x 1 1/2	345 1/2	319.8
2	20 x 7 1/2	16 x 1 1/2	317 1/2	389.0	3	12 x 6a	20 x 1 1/2	403 1/2	315.9
3	15 x 6	21 x 1 1/2	395	380.5	1	18 x 7	12 x 2	242	310.5
3	18 x 7	24 x 1 1/2	353	376.2	2	15 x 6	14 x 2	311	309.9
3	14 x 6a	20 x 1 1/2	412 1/2	374.7	2	20 x 7 1/2	16 x 1 1/2	263 1/2	307.6
1	18 x 7	12 x 2 1/2	283	373.1	2	16 x 6	14 x 1 1/2	293	302.5
2	24 x 7 1/2	16 x 1 1/2	271 1/2	369.9	3	14 x 6a	20 x 1 1/2	344 1/2	301.3
3	16 x 6	21 x 1 1/2	368	365.1	3	16 x 6	21 x 1 1/2	314 1/2	299.9
2	18 x 7	16 x 1 1/2	317	362.2	3	15 x 5	18 x 1 1/2	313	299.2
1	24 x 7 1/2	12 x 1 1/2	226	361.1	3	15 x 6	21 x 1	323 1/2	298.2
1	20 x 7 1/2	12 x 2	256	358.0	1	20 x 7 1/2	12 x 1 1/2	215	290.5
3	18 x 7	24 x 1 1/2	332 1/2	348.8	2	20 x 7 1/2	16 x 1 1/2	249 1/2	287.5
3	12 x 6a	20 x 2	437 1/2	348.5	2	18 x 7	16 x 1	262 1/2	287.5
2	20 x 7 1/2	16 x 1	290 1/2	348.1	2	14 x 6a	14 x 2	307	286.8
2	24 x 7 1/2	16 x 1 1/2	258	345.8	3	12 x 6a	20 x 1 1/2	369 1/2	283.8
1	18 x 7	12 x 2 1/2	262 1/2	341.5	3	14 x 6b	20 x 1 1/2	311 1/2	282.8

In this table the single, double, and triple joist-compound girders are brought together and arranged in descending order of strength. The method of selection of a suitable girder for any system of loading is as follows:—

Calculate Maximum Bending Moment in foot tons. See Part IV.

Refer to column headed "Maximum Moment of Resistance, foot tons."

Any girder of which the Maximum Moment of Resistance is not less than the calculated Bending Moment will have sufficient carrying capacity.

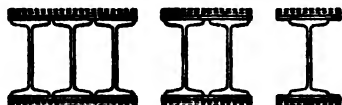
Safe working stress = 7.5 tons per square inch.

Note deflection, web-buckling, and rivet-pitch limitations.

For safe loads and properties of these girders, see pages 20 to 49.

For full explanations of tables, see notes commencing page 108.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND GIRDERS.

Arranged in Descending Order of Carrying Capacity.

Composed of			Weight per foot in lbs.	Maximum Moment of Resistance in foot tons.	Composed of			Weight per foot in lbs.	Maximum Moment of Resistance in foot tons.
1 2 3	Steel Joist(s).	Plates, each Flange to form.			1 2 3	Steel Joist(s).	Plates, each Flange to form.		
1	24 × 7½	12 × 1	185½	281.5	3	12 × 6b	20 × 1½	305½	239.5
2	15 × 6	14 × 1½	287	281.5	3	15 × 6	21 × 1	270	238.0
1	18 × 7	12 × 1½	221½	279.6	3	16 × 6	21 × ½	261	235.9
3	16 × 6	21 × ½	296½	278.5	2	14 × 6a	14 × 1½	259½	234.0
3	15 × 6	21 × ½	305½	278.0	2	18 × 7	16 × ½	222	232.5
2	16 × 6	14 × 1½	269	272.7	3	14 × 6a	20 × ½	276½	230.1
3	12 × 6b	20 × 1½	339½	270.7	3	15 × 5	18 × 1	252	229.2
2	18 × 7	16 × ½	249	269.1	3	14 × 6b	20 × ½	260½	228.3
2	20 × 7½	16 × ½	236	267.5	1	16 × 6	10 × 2	200½	227.3
3	14 × 6a	20 × 1	310½	265.4	2	15 × 6	14 × 1½	239½	226.0
3	15 × 5	18 × 1½	282½	264.0	3	12 × 5	18 × 1½	283	225.7
1	24 × 7½	12 × 1½	175	261.8	1	20 × 7½	12 × 1	174½	224.2
2	14 × 6a	14 × 1½	283	260.2	1	24 × 7½	12 × ½	155	222.5
3	15 × 6	21 × ½	287½	257.9	3	10 × 6	20 × 1½	333½	222.2
1	20 × 7½	12 × 1½	195	257.2	3	12 × 6a	20 × 1	301½	221.7
3	16 × 6	21 × ½	279	257.1	2	14 × 6b	14 × 1½	237½	220.8
2	15 × 6	14 × 1½	263½	253.5	2	12 × 6a	14 × 1½	277	219.5
3	12 × 6a	20 × 1½	335½	253.2	1	18 × 7	12 × 1½	181	218.9
2	18 × 7	16 × ½	235½	250.8	3	15 × 6	21 × ½	252	218.1
1	18 × 7	12 × 1½	201	249.1	2	18 × 7	16 × ½	208½	214.3
3	14 × 6a	20 × ½	293½	247.7	2	16 × 6	14 × 1	221½	214.3
3	14 × 6b	20 × 1	277½	246.4	3	14 × 6a	20 × ½	259½	212.5
2	16 × 6	14 × 1½	245½	243.3	1	15 × 6	10 × 2	197½	212.2
2	12 × 6a	14 × 2	301	242.6	3	15 × 5	18 × ½	236½	211.9
1	24 × 7½	12 × ¾	165	242.1	3	14 × 6b	20 × ¾	243½	210.4

In this table the single, double, and triple joist-compound girders are brought together and arranged in descending order of strength. The method of selection of a suitable girder for any system of loading is as follows:—

Calculate Maximum Bending Moment in foot tons. See Part IV.

Refer to column headed "Maximum Moment of Resistance, foot tons."

Any girder of which the Maximum Moment of Resistance is not less than the calculated Bending Moment will have sufficient carrying capacity.

Safe working stress = 7.5 tons per square inch.

Note deflection, web-buckling, and rivet pitch limitations.

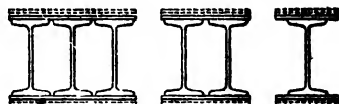
For safe loads and properties of these girders, see pages 20 to 49.

For full explanations of tables, see notes commencing page 108.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND GIRDERS.

Arranged in Descending Order of Carrying Capacity.



Composed of			Weight per foot in lbs.	Maximum Moment of Resistance in foot tons.	Composed of			Weight per foot in lbs.	Maximum Moment of Resistance in foot tons.
1 2 3	Steel Joist(s).	Plates, each Flange to form.			1 2 3	Steel Joist(s).	Plates, each Flange to form.		
2	14 x 6a	14 x 1 1/2	235 1/2	208.2	2	15 x 6	14 x 7/8	204	185.3
1	20 x 7 1/2	12 x 7/8	164	207.7	2	14 x 6a	14 x 1	211 1/2	182.8
3	12 x 6b	20 x 1	271 1/2	207.5	1	16 x 6	10 x 1 1/2	166 1/2	181.8
3	12 x 6a	20 x 1	284 1/2	206.5	3	14 x 6a	20 x 1 1/2	225 1/2	177.9
1	16 x 6	10 x 1 1/2	183 1/2	204.4	3	15 x 5	18 x 1 1/2	206	177.7
1	24 x 7 1/2	12 x 1 1/2	144 1/2	202.9	1	14 x 6a	10 x 1 1/2	178 1/2	176.8
2	16 x 6	14 x 1	209 1/2	199.9	3	12 x 6b	20 x 1 1/2	237 1/2	176.7
2	15 x 5	12 x 1 1/2	209	199.5	3	12 x 6a	20 x 1 1/2	220 1/2	176.5
2	15 x 6	14 x 1	215 1/2	198.8	2	15 x 5	12 x 1 1/2	188 1/2	176.0
3	12 x 5	18 x 1 1/2	252 1/2	197.7	2	12 x 6a	14 x 1 1/2	229 1/2	175.1
1	14 x 6a	10 x 2	195 1/2	197.0	1	20 x 7 1/2	12 x 1 1/2	144	175.1
2	12 x 6a	14 x 1 1/2	253 1/2	196.9	3	14 x 6b	20 x 1	209 1/2	174.9
2	14 x 6b	14 x 1 1/2	213 1/2	195.9	1	18 x 7	12 x 7/8	150	174.0
3	10 x 6	20 x 1 1/2	299 1/2	195.4	2	15 x 6	14 x 1	192	171.9
3	14 x 6a	20 x 1 1/2	242 1/2	195.2	2	16 x 6	14 x 1	186	171.4
3	15 x 5	18 x 1 1/2	221 1/2	194.8	2	14 x 6a	14 x 1	200	170.2
3	14 x 6b	20 x 1 1/2	226 1/2	192.5	2	14 x 6b	14 x 1	189 1/2	170.1
3	12 x 6b	20 x 1	254 1/2	192.1	1	15 x 6	10 x 1 1/2	163 1/2	169.4
3	12 x 6a	20 x 1	267 1/2	191.4	3	8 x 6	20 x 1 1/2	312 1/2	169.4
1	20 x 7 1/2	12 x 1	154	191.4	3	10 x 6	20 x 1	265 1/2	169.2
1	15 x 6	10 x 1 1/2	180 1/2	190.7	3	12 x 5	18 x 1	222	168.8
1	18 x 7	12 x 1	160 1/2	188.9	1	12 x 6a	10 x 2	192 1/2	167.6
2	12 x 6b	14 x 1 1/2	233 1/2	188.0	2	12 x 6b	14 x 1 1/2	209 1/2	166.0
2	16 x 6	14 x 1	198	185.6	3	12 x 6a	20 x 1 1/2	233 1/2	161.7
3	10 x 5	18 x 1 1/2	277	185.6	3	12 x 6b	20 x 1 1/2	220 1/2	161.6

In this table the single, double, and triple joist-compound girders are brought together and arranged in descending order of strength. The method of selection of a suitable girder for any system of loading is as follows:—

Calculate Maximum Bending Moment in foot tons. See Part IV.

Refer to column headed "Maximum Moment of Resistance, foot tons."

Any girder of which the Maximum Moment of Resistance is not less than the calculated Bending Moment will have sufficient carrying capacity.

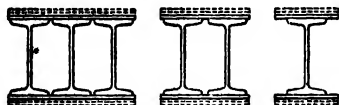
Safe working stress = 7.5 tons per square inch.

Note deflection, web-buckling, and rivet-pitch limitations.

For safe loads and properties of these girders, see pages 20 to 49.

For full explanations of tables, see notes commencing page 108.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND GIRDERS.

Arranged in Descending Order of Carrying Capacity.

Composed of			Weight per foot in lbs.	Maximum Moment of Resistance in foot tons.	Composed of			Weight per foot in lbs.	Maximum Moment of Resistance in foot tons.
1 2 3	Steel Joist(s).	Plates, each Flange to form.			1 2 3	Steel Joist(s).	Plates, each Flange to form.		
3	10 x 5	18 x 1 1/2	246 1/2	161.4	2	14 x 6b	14 x 1 1/2	166	144.7
3	15 x 5	18 x 1 1/2	191	160.7	1	18 x 7	12 x 1 1/2	130	144.4
1	16 x 6	10 x 1 1/2	149 1/2	159.4	3	15 x 5	18 x 1 1/2	175 1/2	143.9
1	18 x 7	12 x 1 1/2	140	159.2	3	10 x 6	20 x 1 1/2	231 1/2	143.7
1	20 x 7 1/2	12 x 1 1/2	133 1/2	158.8	2	12 x 6b	14 x 1 1/2	185 1/2	143.4
2	15 x 6	14 x 1 1/2	180	158.6	1	15 x 5	9 x 1 1/2	136	142.4
2	14 x 6a	14 x 1 1/2	188	157.7	2	12 x 6a	14 x 1 1/2	194	142.1
2	14 x 6b	14 x 1 1/2	178 1/2	157.4	2	15 x 5	12 x 1 1/2	158	141.3
2	16 x 6	14 x 1 1/2	174	157.3	3	12 x 5	18 x 1 1/2	191 1/2	141.0
3	14 x 6b	20 x 1 1/2	192 1/2	157.2	3	10 x 5	18 x 1 1/2	216	137.7
1	14 x 6a	10 x 1 1/2	161 1/2	156.9	1	16 x 6	10 x 1 1/2	132 1/2	137.3
3	10 x 6	20 x 1 1/2	248 1/2	156.4	1	14 x 6a	10 x 1 1/2	144 1/2	137.2
3	12 x 5	18 x 1 1/2	206 1/2	154.9	2	10 x 6	14 x 1 1/2	205 1/2	135.6
2	10 x 6	14 x 1 1/2	229 1/2	154.6	2	14 x 6a	14 x 1 1/2	164 1/2	133.1
2	15 x 5	12 x 1 1/2	168	152.8	1	12 x 6a	10 x 1 1/2	158 1/2	132.7
2	12 x 6a	14 x 1 1/2	205 1/2	152.8	2	12 x 6b	14 x 1 1/2	174	132.5
1	14 x 6b	10 x 1 1/2	150 1/2	150.9	2	14 x 6b	14 x 1 1/2	154	132.0
2	12 x 5	12 x 1 1/2	189	150.5	2	12 x 5	12 x 1 1/2	168 1/2	131.8
1	12 x 6a	10 x 1 1/2	175 1/2	150.0	3	12 x 6b	20 x 1 1/2	180 1/2	131.6
1	15 x 6	10 x 1 1/2	146 1/2	148.4	2	12 x 6a	14 x 1 1/2	182	131.4
3	8 x 6	20 x 1 1/2	278 1/2	147.3	1	14 x 6b	10 x 1 1/2	133 1/2	131.1
3	12 x 6a	20 x 1 1/2	216 1/2	147.0	3	10 x 6	20 x 1 1/2	214 1/2	131.1
3	12 x 6b	20 x 1 1/2	203 1/2	146.5	2	15 x 5	12 x 1 1/2	147 1/2	129.9
2	15 x 6	14 x 1 1/2	168	145.4	1	18 x 7	12 x 1 1/2	119 1/2	129.7
2	14 x 6a	14 x 1 1/2	176	145.4	1	12 x 6b	10 x 1 1/2	148 1/2	128.3

In this table the single, double, and triple joist-compound girders are brought together and arranged in descending order of strength. The method of selection of a suitable girder for any system of loading is as follows:—

Calculate Maximum Bending Moment in foot tons. See Part IV.

Refer to column headed "Maximum Moment of Resistance, foot tons."

Any girder of which the Maximum Moment of Resistance is not less than the calculated Bending Moment will have sufficient carrying capacity.

Safe working stress = 7.5 tons per square inch.

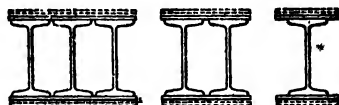
Note deflection, web-buckling, and rivet-pitch limitations.

For safe loads and properties of these girders, see pages 20 to 49.

For full explanations of tables, see notes commencing page 108.

# COMPOUND GIRDERS.

Arranged in Descending Order of Carrying Capacity.



Composed of			Weight per foot in lbs.	Maximum Moment of Resistance in foot tons.	Composed of			Weight per foot in lbs.	Maximum Moment of Resistance in foot tons.
1 2 3	Steel Joist(s).	Plates, each Flange to form.			1 2 3	Steel Joist(s).	Plates, each Flange to form.		
1	15 x 6	10 x 1	129½	127.7	2	12 x 6b	14 x 1	150	110.9
3	12 x 5	18 x 1	176	127.2	2	12 x 6a	14 x 1	158	110.3
1	16 x 6	10 x 1	124	126.3	1	12 x 5	9 x 1	126½	109.6
3	10 x 5	18 x 1	200½	126.0	1	14 x 6a	10 x 1	119	108.2
3	8 x 6	20 x 1	244½	125.9	2	10 x 6	14 x 1	170	107.9
2	10 x 5	12 x 1½	185	123.7	3	8 x 5	18 x 1	210	107.8
1	15 x 5	9 x 1½	121	123.4	2	10 x 5	12 x 1½	164½	107.6
2	12 x 6b	14 x 1	162	121.6	2	15 x 5	12 x 1	127½	107.2
2	12 x 6a	14 x 1	170	120.8	1	15 x 6	10 x 1	112½	107.1
2	14 x 6a	14 x 1	152	120.7	2	14 x 6b	14 x 1	130	107.0
2	14 x 6b	14 x 1	142	119.5	3	10 x 6	20 x 1	180½	106.3
3	10 x 6	20 x 1	197½	118.6	1	10 x 6	10 x 1	146½	106.0
2	15 x 5	12 x 1½	137½	118.5	3	8 x 6	20 x 1	210½	105.0
2	8 x 6	14 x 1½	215½	118.2	1	15 x 5	9 x 1	105½	104.7
1	14 x 6a	10 x 1	127½	117.8	1	16 x 6	10 x 1	107	104.5
1	15 x 6	10 x 1	121	117.4	2	12 x 5	12 x 1	138	103.2
2	10 x 6	14 x 1	181½	117.1	3	10 x 5	18 x 1	170	103.0
1	12 x 6a	10 x 1½	141½	116.1	2	8 x 6	14 x 1½	191½	102.5
1	16 x 6	10 x 1	115½	115.4	1	14 x 6b	10 x 1	108	101.7
3	8 x 6	20 x 1	227½	115.4	2	12 x 6b	14 x 1	138	100.2
3	10 x 5	18 x 1	185½	114.4	3	12 x 5	18 x 1	145½	100.0
3	12 x 5	18 x 1	161	113.6	2	12 x 6a	14 x 1	146	99.9
2	12 x 5	12 x 1	148	112.6	1	12 x 6a	10 x 1	125	99.1
1	12 x 6b	10 x 1½	131½	111.6	2	10 x 6	14 x 1	158	98.9
1	14 x 6b	10 x 1	116½	111.4	1	14 x 6a	10 x 1	110½	98.6

In this table the single, double, and triple joist-compound girders are brought together and arranged in descending order of strength. The method of selection of a suitable girder for any system of loading is as follows:—

Calculate Maximum Bending Moment in foot tons. See Part IV.

Refer to column headed "Maximum Moment of Resistance, foot tons."

Any girder of which the Maximum Moment of Resistance is not less than the calculated Bending Moment will have sufficient carrying capacity.

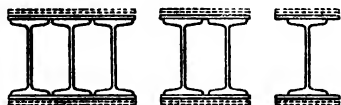
Safe working stress=7.5 tons per square inch.

Note deflection, web-buckling, and rivet-pitch limitations.

For safe loads and properties of these girders, see pages 20 to 49.

For full explanations of tables, see notes commencing page 108.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND GIRDERS.

Arranged in Descending Order of Carrying Capacity.

Composed of			Weight per foot in lbs.	Maximum Moment of Resistance in foot tons.	Composed of			Weight per foot in lbs.	Maximum Moment of Resistance in foot tons.
1 2 3	Steel Joist(s).	Plates, each Flange to form.			1 2 3	Steel Joist(s).	Plates, each Flange to form.		
3	8 x 5	18 x 7	195	98.4	2	10 x 5	12 x 7	134	84.0
1	15 x 6	10 x 6	104	96.9	1	12 x 6a	10 x 6	107½	82.6
2	15 x 5	12 x 6	117	95.9	1	14 x 6b	10 x 6	91	82.4
1	15 x 5	9 x 6	98	95.4	1	8 x 6	10 x 1	139½	82.3
3	8 x 6	20 x 6	193½	94.8	2	10 x 6	14 x 6	134	81.2
1	12 x 5	9 x 1½	111	94.5	3	10 x 5	18 x 6	139½	80.4
1	12 x 6b	10 x 1	114½	94.4	3	8 x 5	18 x 6	164	80.0
2	12 x 5	12 x 6	127½	94.0	2	8 x 6	14 x 6	156	79.9
1	16 x 6	10 x 6	98½	93.6	1	14 x 6a	10 x 6	93½	79.6
1	14 x 6b	10 x 6	99½	92.1	1	12 x 5	9 x 1	95½	78.9
2	10 x 5	12 x 1	144	91.7	1	12 x 6b	10 x 6	97½	77.7
3	10 x 5	18 x 6	155	91.7	1	10 x 5	9 x 1½	109	77.6
1	10 x 6	10 x 1½	129½	91.6	1	10 x 6	10 x 1	112½	77.5
1	12 x 6a	10 x 6	116	90.8	1	15 x 5	9 x 6	82½	76.9
1	10 x 5	9 x 1½	124½	90.6	2	10 x 5	12 x 6	123½	76.3
2	10 x 6	14 x 6	146	90.0	2	12 x 5	12 x 6	107½	75.7
2	12 x 6b	14 x 6	126	89.6	3	8 x 6	20 x 6	159½	74.9
1	14 x 6a	10 x 6	102	89.1	1	12 x 6a	10 x 9	99	74.5
3	8 x 5	18 x 6	179½	89.1	1	14 x 6b	10 x 6	82½	72.9
2	8 x 6	14 x 1	167½	87.3	2	8 x 6	14 x 6	144	72.6
1	15 x 6	10 x 6	95½	86.8	2	10 x 6	14 x 6	122	72.4
1	15 x 5	9 x 6	90½	86.1	2	8 x 5	12 x 1	140	71.9
1	12 x 6b	10 x 6	106	86.1	1	12 x 5	9 x 6	88	71.4
2	12 x 5	12 x 6	117½	84.8	3	8 x 5	18 x 6	149	70.9
3	8 x 6	20 x 6	176½	84.7	1	10 x 6	10 x 6	104	70.5

In this table the single, double, and triple joist-compound girders are brought together and arranged in descending order of strength. The method of selection of a suitable girder for any system of loading is as follows:—

Calculate Maximum Bending Moment in foot tons. See Part IV.

Refer to column headed "Maximum Moment of Resistance, foot tons."

Any girder of which the Maximum Moment of Resistance is not less than the calculated Bending Moment will have sufficient carrying capacity.

Safe working stress = 7.5 tons per square inch.

Note deflection, web-buckling, and rivet-pitch limitations.

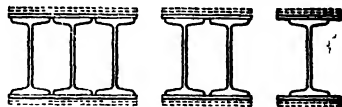
For safe loads and properties of these girders, see pages 20 to 49.

For full explanations of tables, see notes commencing page 108.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND GIRDERS.

Arranged in Descending Order of Carrying Capacity.



Composed of			Weight per foot in lbs.	Maximum Moment of Resistance in foot tons.	Composed of			Weight per foot in lbs.	Maximum Moment of Resistance in foot tons.
1 2 3	Steel Joist(s).	Plates, each Flange to form.			1 2 3	Steel Joist(s).	Plates, each Flange to form.		
1	8 x 6	10 x 1 1/2	122 1/2	70.4	2	8 x 5	12 x 1 1/2	109 1/2	53.3
1	14 x 6a	10 x 1 1/2	85	70.2	1	12 x 6b	10 x 1 1/2	72	53.2
1	12 x 6b	10 x 1 1/2	89	69.6	1	8 x 6	10 x 1 1/2	97	53.2
2	10 x 5	12 x 1 1/2	113 1/2	68.7	1	10 x 5	9 x 1 1/2	78 1/2	52.3
1	15 x 5	9 x 1 1/2	75	67.7	1	8 x 5	9 x 1 1/2	91 1/2	51.2
2	12 x 5	12 x 1 1/2	97	66.7	2	8 x 6	14 x 1 1/2	108	51.1
1	12 x 6a	10 x 1 1/2	90 1/2	66.4	1	10 x 6	10 x 1 1/2	78 1/2	50.0
2	8 x 5	12 x 1 1/2	130	65.6	1	12 x 5	9 x 1 1/2	65	49.1
2	8 x 6	14 x 1 1/2	132	65.3	1	8 x 6	10 x 1 1/2	88 1/2	47.6
1	10 x 5	9 x 1 1/2	93 1/2	64.9	2	8 x 5	12 x 1 1/2	99 1/2	47.3
1	12 x 5	9 x 1 1/2	80 1/2	63.9	1	10 x 5	9 x 1 1/2	70 1/2	46.1
1	10 x 6	10 x 1 1/2	95 1/2	63.6	1	8 x 5	9 x 1 1/2	83 1/2	46.1
1	14 x 6b	10 x 1 1/2	74	63.4	1	10 x 6	10 x 1 1/2	70	43.2
3	8 x 5	18 x 1 1/2	133 1/2	62.1	1	8 x 6	10 x 1 1/2	80	42.1
1	12 x 6b	10 x 1 1/2	80 1/2	61.4	1	12 x 5	9 x 1 1/2	57 1/2	41.7
2	10 x 5	12 x 1 1/2	103 1/2	61.1	2	8 x 5	12 x 1 1/2	89	41.4
2	8 x 5	12 x 1 1/2	119 1/2	59.4	1	8 x 5	9 x 1 1/2	76 1/2	41.1
1	8 x 6	10 x 1 1/2	107 1/2	58.9	1	10 x 5	9 x 1 1/2	63	39.9
1	10 x 5	9 x 1 1/2	86	58.6	1	8 x 6	10 x 1 1/2	71 1/2	36.6
1	15 x 5	9 x 1 1/2	67 1/2	58.5	1	8 x 5	9 x 1 1/2	69	36.1
1	12 x 6a	10 x 1 1/2	82	58.4	1	10 x 5	9 x 1 1/2	55 1/2	33.9
2	8 x 6	14 x 1 1/2	120	58.2	1	8 x 6	10 x 1 1/2	63	31.2
1	10 x 6	10 x 1 1/2	87	56.7	1	8 x 5	9 x 1 1/2	61	31.2
1	12 x 5	9 x 1 1/2	72 1/2	56.5	1	8 x 5	9 x 1 1/2	53 1/2	26.3
2	10 x 5	12 x 1 1/2	93	53.6					

In this table the single, double, and triple joist-compound girders are brought together and arranged in descending order of strength. The method of selection of a suitable girder for any system of loading is as follows:—

Calculate Maximum Bending Moment in foot tons. See Part IV.

Refer to column headed "Maximum Moment of Resistance, foot tons."

Any girder of which the Maximum Moment of Resistance is not less than the calculated Bending Moment will have sufficient carrying capacity.

Safe working stress = 7.5 tons per square inch.

Note deflection, web-buckling, and rivet-pitch limitations.

For safe loads and properties of these girders, see pages 20 to 49.

For full explanations of tables, see notes commencing page 108.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND GIRDERS.

Safe Distributed Loads, in Tons.

Reference Mark.	Size, D x B inches.	SPANS IN FEET.															
		6	8	10	12	14	16	18	20	22	24	26	28	30	32		
29D	16 $\frac{1}{2}$ x 12	84.6	63.4	50.7	42.3	36.2	31.7	28.2	25.4	23.0	21.1	19.5	18.1	16.9	15.8		
28D	16 $\frac{1}{2}$ x "	82.7	62.0	49.6	41.3	35.4	31.0	27.5	24.8	22.5	20.7	19.1	17.7	16.5	15.5		
27D	15 $\frac{1}{2}$ x 12	77.6	58.2	46.6	38.8	33.3	29.1	25.9	23.3	21.2	19.4	17.9	16.6	15.5	14.5		
26D	15 $\frac{1}{2}$ x "	75.9	56.9	45.5	37.9	32.5	28.5	25.3	22.8	20.7	19.0	17.5	16.3	15.2	14.2		
25D	15 $\frac{1}{2}$ x 10	54.1	40.5	32.4	27.0	23.2	20.3	18.0	16.2	14.7	13.5	12.5	11.6	10.8	10.1		
24D	15 $\frac{1}{2}$ x "	52.8	39.6	31.7	26.4	22.6	19.8	17.6	15.8	14.4	13.2	12.2	11.3	10.6	9.9		
23D	15 $\frac{1}{2}$ x "	51.3	38.5	30.8	25.6	22.0	19.2	17.1	15.4	14.0	12.8	11.8	11.0	10.3	9.6		
22D	14 $\frac{1}{2}$ x 12	70.6	52.9	42.3	35.3	30.2	26.5	23.5	21.2	19.2	17.6	16.3	15.1	14.1	13.2		
21D	14 $\frac{1}{2}$ x "	68.5	51.4	41.1	34.2	29.3	25.7	22.8	20.5	18.7	17.1	15.8	14.7	13.7	12.8		
20D	14 $\frac{1}{2}$ x 12	58.7	44.1	35.2	29.4	25.1	22.0	19.6	17.6	16.0	14.7	13.6	12.6	11.7	11.0		
19D	14 $\frac{1}{2}$ x "	57.6	43.2	34.5	28.8	24.7	21.6	19.2	17.3	15.7	14.4	13.3	12.3	11.5	10.8		
18D	12 $\frac{1}{2}$ x 12	58.1	43.6	34.9	29.1	24.9	21.8	19.4	17.4	15.8	14.5	13.4	12.5	11.6	10.9		
17D	12 $\frac{1}{2}$ x "	57.0	42.7	34.2	28.5	24.4	21.4	19.0	17.1	15.5	14.2	13.1	12.2	11.4	10.7		
16D	12 $\frac{1}{2}$ x 12	49.1	36.8	29.4	24.5	21.0	18.4	16.4	14.7	13.4	12.3	11.3	10.5	9.8	9.2		
15D	12 $\frac{1}{2}$ x "	48.0	36.0	28.8	24.0	20.6	18.0	16.0	14.4	13.1	12.0	11.1	10.3	9.6	9.0		
14D	12 $\frac{1}{2}$ x "	46.9	35.2	28.1	23.4	20.1	17.6	15.6	14.1	12.8	11.7	10.8	10.0	9.4	8.8		
13D	12 $\frac{1}{2}$ x 10	35.0	26.2	21.0	17.5	15.0	13.1	11.7	10.5	9.5	8.7	8.1	7.5	7.0	6.6		
12D	12 $\frac{1}{2}$ x "	34.2	25.5	20.5	17.1	14.6	12.8	11.4	10.2	9.3	8.5	7.9	7.3	6.8	6.4		
11D	12 $\frac{1}{2}$ x "	33.2	24.9	20.0	16.6	14.2	12.5	11.1	10.0	9.0	8.3	7.7	7.1	6.6	6.2		
10D	10 $\frac{1}{2}$ x 12	39.7	29.7	23.8	19.8	17.0	14.9	13.2	11.9	10.8	9.9	9.1	8.5	7.9	7.4		
9D	10 $\frac{1}{2}$ x "	38.8	29.1	23.3	19.4	16.6	14.6	12.9	11.6	10.6	9.7	9.0	8.3	7.8			
8D	10 $\frac{1}{2}$ x 10	27.9	20.9	16.7	14.0	12.0	10.5	9.3	8.4	7.6	7.0	6.4	6.0	5.6	5.2		
7D	10 $\frac{1}{2}$ x "	27.2	20.4	16.3	13.6	11.7	10.2	9.1	8.2	7.4	6.8	6.3	5.8	5.4			
6D	10 $\frac{1}{2}$ x "	26.5	19.9	15.9	13.2	11.3	9.9	8.8	7.9	7.2	6.6	6.1	5.7	5.3			
5D	8 $\frac{1}{2}$ x 12	26.7	20.0	16.0	13.3	11.4	10.0	8.9	8.0	7.3	6.7	6.2					
4D	8 $\frac{1}{2}$ x "	26.1	19.6	15.6	13.0	11.2	9.8	8.7	7.8	7.1	6.5	6.0					
3D	8 $\frac{1}{2}$ x 10	21.4	16.1	12.8	10.7	9.2	8.0	7.1	6.4	5.8	5.3	4.9					
2D	8 $\frac{1}{2}$ x "	20.9	15.7	12.5	10.4	8.9	7.8	7.0	6.3	5.7	5.2	4.8					
1D	8 $\frac{1}{2}$ x "	20.3	15.2	12.2	10.2	8.7	7.6	6.8	6.1	5.5	5.1						

Rivets  $\frac{3}{4}$ -in.  
diam. at  
6-in. pitch

Tabular loads to right of full zigzag line will produce deflection greater than  $\frac{1}{26}$ th of an inch per foot of span.

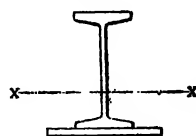
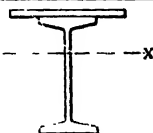
Girders supporting full tabular loads to left of dotted zigzag line require stiffeners to prevent web buckling.

Safe working stress = 7.5 tons per square inch, equal to a factor of safety of 4. Ends of girders simply supported.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND GIRDERS.

Composition and Properties.



Composed of		Weight per foot in lbs.	Area in square inches.	Maxi- mum Moment of Inertia. X-X	Maxi- mum Modulus of Section. X-X	Safe Distributed Load on 1 Foot Span for		Deflection Coefficient. X-X
One Steel Joist.	One Flange Plate.					Girder.	1 in. Plate width.	
16 x 6	12 x $\frac{5}{16}$	89	25.7	1058	101.5	507	10.4	.000900
"	" x $\frac{5}{16}$	83 $\frac{1}{2}$	24.2	997	99.2	496	9.6	.000934
15 x 6	12 x $\frac{5}{16}$	86	24.8	918	93.2	466	9.4	.000951
"	" x $\frac{5}{16}$	81	23.3	865	91.1	455	8.7	.000987
15 x 5	10 x $\frac{5}{16}$	64 $\frac{1}{2}$	18.6	657	64.9	324	8.3	.000926
"	" x $\frac{5}{16}$	60 $\frac{1}{2}$	17.3	617	63.4	317	7.8	.000964
"	" x $\frac{5}{16}$	56	16.1	572	61.6	308	7.0	.001009
14 x 6a	12 x $\frac{5}{16}$	84	24.3	784	84.7	423	8.6	.001012
"	" x $\frac{5}{16}$	79	22.8	740	82.2	411	7.6	.001042
14 x 6b	12 x $\frac{5}{16}$	73	21.0	678	70.5	352	7.1	.000974
"	" x $\frac{5}{16}$	68	19.5	638	69.1	345	6.8	.001016
12 x 6a	12 x $\frac{5}{16}$	81	23.4	560	69.8	349	7.1	.001168
"	" x $\frac{5}{16}$	76	21.9	527	68.4	342	6.7	.001217
12 x 6b	12 x $\frac{5}{16}$	71	20.4	491	58.9	294	5.9	.001126
"	" x $\frac{5}{16}$	66	18.9	461	57.6	288	5.6	.001172
"	" x $\frac{5}{16}$	60 $\frac{1}{2}$	17.4	428	56.3	281	5.2	.001234
12 x 5	10 x $\frac{5}{16}$	54 $\frac{1}{2}$	15.7	358	42.0	210	5.2	.001100
"	" x $\frac{5}{16}$	50 $\frac{1}{2}$	14.4	335	41.0	205	5.1	.001147
"	" x $\frac{5}{16}$	46	13.2	310	39.9	199	4.7	.001207
10 x 6	12 x $\frac{5}{16}$	69	19.9	334	47.6	238	4.9	.001338
"	" x $\frac{5}{16}$	64	18.4	313	46.6	233	4.6	.001395
10 x 5	10 x $\frac{5}{16}$	52 $\frac{1}{2}$	15.1	241	33.5	167	4.2	.001301
"	" x $\frac{5}{16}$	48 $\frac{1}{2}$	13.8	225	32.7	163	4.1	.001360
"	" x $\frac{5}{16}$	44	12.6	208	31.8	159	3.8	.001432
8 x 6	12 x $\frac{5}{16}$	62	17.8	186	32.0	160	3.3	.001611
"	" x $\frac{5}{16}$	57	16.3	174	31.3	156	3.2	.001685
8 x 5	10 x $\frac{5}{16}$	50 $\frac{1}{2}$	14.5	161	25.7	128	3.2	.001600
"	" x $\frac{5}{16}$	46 $\frac{1}{2}$	13.2	141	25.1	125	3.1	.001672
"	" x $\frac{5}{16}$	42	12.0	129	24.4	122	2.9	.001766

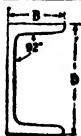
In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

Let  $\delta$  = deflection, K = deflection coefficient, and L = span in feet, then  $\delta = K \times L^2$ .

For full explanations of tables, see notes commencing page 108.

For formulae, explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.



## STEEL CHANNELS.

Safe Distributed Loads, in Tons.

Reference Mark.	Size, D x B inches.	SPANS IN FEET.															
		4	6	8	10	12	14	16	18	20	22	24	26	28	30	32	
BSC27	*15 x 4	62.8	41.9	31.4	25.1	20.9	17.9	15.7	13.9	12.5	11.4	10.5	9.6	9.0	8.4	7.8	
BSC26	12 x 4	45.4	30.3	22.7	18.2	15.1	13.0	11.3	10.1	9.1	8.2	7.6	7.0	6.5			
BSC25	*12 x 3½	39.7	26.5	19.9	15.9	13.2	11.3	9.9	8.8	7.9	7.2	6.6	6.1	5.7			
BSC24	*12 x 3½	33.0	22.0	16.5	13.2	11.0	9.4	8.2	7.3	6.6	6.0	5.5	5.1	4.7			
BSC23	11 x 4	38.7	25.8	19.4	15.5	12.9	11.0	9.7	8.6	7.7	7.0	6.4	5.9				
BSC22	11 x 3½	33.8	22.5	16.9	13.5	11.3	9.6	8.4	7.5	6.7	6.1	5.6	5.2				
BSC21	10 x 4	32.7	21.8	16.3	13.1	10.9	9.3	8.1	7.2	6.5	5.9	5.4					
BSC20	*10 x 3½	29.5	19.6	14.7	11.8	9.8	8.4	7.4	6.5	5.9	5.3	4.9					
BSC19	*10 x 3½	25.6	17.1	12.8	10.3	8.5	7.3	6.4	5.7	5.1	4.6	4.3					
BSC18	9 x 4	28.2	18.6	14.1	11.3	9.4	8.0	7.0	6.2	5.6	5.1						
BSC17	*9 x 3½	24.4	16.3	12.2	9.8	8.1	7.0	6.1	5.4	4.9	4.4						
BSC16	*9 x 3½	22.2	14.8	11.1	8.9	7.4	6.3	5.5	4.9	4.4	4.0						
BSC15	9 x 3	18.1	12.0	9.0	7.2	6.0	5.2	4.5	4.0	3.6	3.3						
BSC14	8 x 4	23.1	15.4	11.6	9.2	7.7	6.6	5.8	5.1	4.7							

Tabular loads to right of zigzag line will produce deflection greater than 1/26th of an inch per foot of span.

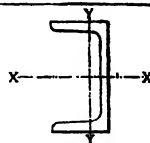
Let  $\delta$  = deflection in inches,  $K$  = deflection coefficient, and  $L$  = span in feet, then  $\delta = K \times L^2$ .

Safe working stress = 7.5 tons per square inch, equal to a factor of safety of 4. Ends of channels simply supported.

# REDPATH, BROWN & CO., LIMITED.

## STEEL CHANNELS.

Dimensions and Properties.



Size. D x B inches.	Weight per foot in lbs.	Area in square inches.	Standard Thicknesses.		Moments of Inertia.		Maxi- mum Modulus of Section. X-X	Safe Distri- buted Load on 1 foot span.	Deflection Coefficient. X-X
			Web.	Flange	Maxi- mum. X-X	Mini- mum. Y-Y			
*15 x 4	41.94	12.334	.525	.630	377.0	14.5	50.2	251.3	.001250
12 x 4	36.47	10.727	.525	.625	218.1	13.6	36.3	181.8	.001562
*12 x 3½	32.88	9.671	.500	.600	190.7	8.9	31.7	158.9	.001562
*12 x 3½	26.10	7.675	.375	.500	158.6	7.5	26.4	132.2	.001562
11 x 4	33.22	9.771	.500	.600	170.4	12.8	30.9	154.9	.001704
11 x 3½	29.82	8.771	.475	.575	148.6	8.4	27.0	135.1	.001704
10 x 4	30.16	8.871	.475	.575	130.7	12.0	26.1	130.7	.001975
*10 x 3½	28.21	8.296	.475	.575	117.9	8.1	23.5	117.9	.001975
*10 x 3½	23.55	6.925	.375	.500	102.6	7.1	20.5	102.6	.001975
9 x 4	28.55	8.396	.475	.575	101.6	11.6	22.5	112.9	.002083
*9 x 3½	25.39	7.469	.450	.550	88.0	7.6	19.5	97.8	.002083
*9 x 3½	22.27	6.550	.375	.500	79.9	6.9	17.7	88.8	.002083
9 x 3	19.27	5.696	.375	.437	65.1	4.0	14.4	72.4	.002083
8 x 4	25.73	7.569	.450	.550	74.0	10.7	18.5	92.5	.002344

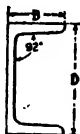
In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

Sections marked (\*) are in our stocks.

For full explanation of tables, see notes commencing page 103. —

For formulae, explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.



## STEEL CHANNELS.

Safe Distributed Loads, in Tons.

Reference Mark.	Size, D x B inches.	SPANS IN FEET.																	
		2	3	4	5	6	7	8	9	10	11	12	14	16	18	20			
BSC13	*8 x 3½	26.6	19.9	15.9	13.3	11.4	10.0	8.8	8.0	7.2	6.6	5.7	5.0	4.4	4.0				
BSC12	8 x 3	22.2	16.7	13.3	11.1	9.5	8.3	7.4	6.7	6.0	5.5	4.8	4.2	3.7	3.3				
BSC11	8 x 2½	17.1	12.8	10.3	8.5	7.3	6.4	5.7	5.1	4.6	4.3	3.7	3.2	2.8	2.5				
BSC10	*7 x 3½	21.2	15.9	12.7	10.6	9.1	7.9	7.0	6.4	5.8	5.3	4.5	4.0	3.5					
BSC 9	7 x 3	17.9	13.4	10.7	8.9	7.7	6.7	5.9	5.4	4.9	4.5	3.8	3.3	3.0					
BSC 8	6 x 3½	16.5	12.3	9.9	8.2	7.0	6.2	5.5	4.9	4.5	4.1	3.5	3.1						
BSC 7	*6 x 3	14.5	10.8	8.7	7.2	6.2	5.4	4.8	4.3	3.9	3.6	3.1	2.7						
BSC 6	*6 x 3	13.3	10.0	8.0	6.7	5.7	5.0	4.4	4.0	3.6	3.3	2.8	2.5						
BSC 5	6 x 2½	15.6	10.4	7.8	6.3	5.2	4.5	3.9	3.5	3.1	2.8	2.6	2.2	1.9					
BSC 4	*5 x 2½	12.1	8.1	6.1	4.9	4.0	3.5	3.0	2.7	2.4	2.2	2.0							
BSC 3	*4 x 2	7.1	4.8	3.6	2.9	2.4	2.0	1.8	1.6	1.4									
BSC 2	3½ x 2	5.3	3.5	2.6	2.1	1.8	1.5	1.3	1.2										
BSC 1	*3 x 1½	3.3	2.2	1.6	1.3	1.1	0.9	0.8											

Tabular loads to right of zigzag line will produce deflection greater than 1/26th of an inch per foot of span.

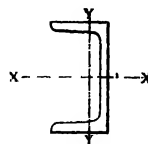
Let  $\delta$  = deflection in inches, K = deflection coefficient, and L = span in feet, then  $\delta = K \times L^2$ .

Safe working stress = 7.5 tons per square inch, equal to a factor of safety of 4. Ends of channels simply supported.

# REDPATH, BROWN & CO., LIMITED.

## STEEL CHANNELS.

### Dimensions and Properties.



Size, D x B inches.	Weight per foot in lbs.	Area in square inches.	Standard Thicknesses.		Moments of Inertia.		Maxi- mum Modulus of Section. X-X	Safe distrib- uted Load on 1 foot Span.	Deflection Coefficient. X-X
			Web.	Flange	Maxi- mum. X-X	Mini- mum. Y-Y			
*8 x 3½	22.72	6.682	.425	.525	63.7	7.0	15.9	79.7	.002344
8 x 3	19.30	5.675	.375	.500	53.4	4.3	13.3	66.7	.002344
8 x 2½	15.12	4.448	.312	.437	41.0	2.2	10.2	51.3	.002344
*7 x 3½	20.23	5.950	.400	.500	44.5	6.4	12.7	63.6	.002679
7 x 3	17.56	5.166	.375	.475	37.6	4.0	10.7	53.7	.002679
6 x 3½	17.90	5.266	.375	.475	29.6	5.9	9.8	49.4	.003125
*6 x 3	16.29	4.791	.375	.475	26.0	3.8	8.6	43.4	.003125
*6 x 3	14.49	4.261	.312	.437	24.0	3.5	8.0	40.0	.003125
6 x 2½	12.04	3.542	.312	.375	18.7	1.8	6.2	31.3	.003125
*5 x 2½	10.98	3.230	.312	.375	12.1	1.7	4.8	24.3	.003750
*4 x 2	7.96	2.341	.250	.375	5.7	0.84	2.8	14.3	.004687
3½ x 2	6.75	1.986	.250	.312	3.7	0.71	2.1	10.6	.005357
*3 x 1½	5.27	1.549	.250	.312	1.9	0.29	1.3	6.6	.006251

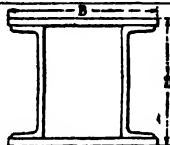
In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

Sections marked (\*) are in our stocks.

For full explanations of tables, see notes commencing page 108.

For formulæ, explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND GIRDERS.

Safe Distributed Loads, in Tons.

Reference Mark.	Size, D x B inches.	SPANS IN FEET.														
		6	8	10	12	14	16	18	20	22	24	26	28	30	32	
29E	16½ x 16						88·3	77·2	68·6	61·8	56·2	51·5	47·5	44·1	41·2	38·6
28E	16½ x "			110	92·2	79·0	69·1	61·5	55·3	50·3	46·1	42·5	39·5	36·9	34·6	
27E	16 x "	122	97·8	81·5	69·8	61·1	54·3	48·9	44·4	40·7	37·6	34·9	32·6	30·5		
26E	16½ x 12			101	84·1	72·1	63·1	56·1	50·5	45·9	42·1	38·8	36·0	33·6	31·5	
25E	16½ x "			115	91·9	76·6	65·6	57·4	51·0	45·9	41·8	38·3	35·3	32·8	30·6	28·7
24E	16 x "	138	103	82·8	69·0	59·1	51·7	46·0	41·4	37·6	34·5	31·8	29·6	27·6	25·9	
23E	13½ x 16						64·9	56·8	50·5	45·4	41·3	37·9	34·9	32·4	30·3	
22E	13½ x "					67·0	57·5	50·3	44·7	40·2	36·6	33·5	30·9	28·7	26·8	
21E	13 x "			87·6	70·1	58·4	50·1	43·8	38·9	35·0	31·8	29·2	26·9	25·0	23·3	
20E	13½ x 12			81·8	65·4	54·5	46·7	40·9	36·3	32·7	29·7	27·3	25·2	23·3	21·8	
19E	13 x "	96·7	72·6	58·1	48·4	41·5	36·3	32·3	29·0	26·4	24·2	22·3	20·7	19·3		
18E	11½ x 16						52·1	45·6	40·5	36·5	33·1	30·4	28·0			
17E	11½ x "						53·5	45·9	40·1	35·7	32·1	29·2	26·7			
16E	11 x "			69·5	55·6	46·3	39·7	34·7	30·9	27·8	25·3	23·1	21·3			
15E	11½ x 12			64·6	51·7	43·1	36·9	32·3	28·7	25·8	23·5	21·5	19·8			
14E	11 x "	75·9	56·9	45·6	37·9	32·5	28·4	25·3	22·8	20·7	18·9	17·5				
13E	10½ x 16						45·8	40·1	35·6	32·1	29·1	26·7				
12E	10½ x "					46·9	40·2	35·2	31·5	28·1	25·5	23·4				
11E	10 x "			48·4	40·4	34·6	30·3	26·9	24·2	22·0	20·2					
10E	10½ x 12			56·2	44·9	37·4	32·1	28·1	25·0	22·5	20·4	18·7				
9E	10 x "	65·7	49·3	39·4	32·9	28·2	24·6	21·9	19·7	17·9	16·2					
8E	9½ x 16					40·6	34·8	30·4	27·0	24·3	22·1					
7E	9 x "					41·7	34·8	29·8	26·1	23·2	20·8	18·9				
6E	9½ x 12			48·3	38·6	32·2	27·6	24·1	21·4	19·3	17·5					
5E	9 x "	56·1	42·1	33·7	28·1	24·1	21·0	18·7	16·8	15·3						
4E	8½ x 16					34·6	29·6	25·9	23·0	20·7						
3E	8 x "			35·4	29·5	25·3	22·1	19·6	17·7							
2E	8½ x 12			40·8	32·6	27·2	23·3	20·4	18·1	16·3						
1E	8 x "	47·2	35·4	28·3	23·6	20·2	17·7	15·7	14·1							

Rivets ½-in.  
diameter.

Rivets ¾-in. diameter.

Tabular loads to right of full zigzag line produce deflection greater than 1/28th of an inch per foot of span.

Girders supporting tabular loads to left of dotted zigzag line require stiffeners to prevent web buckling.

Girders supporting tabular loads printed in ordinary type have rivets at 6 inches pitch.

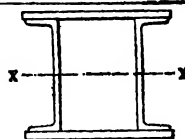
Girders supporting tabular loads printed in italics require a closer pitch of rivets. See page 59.

Safe working stress = 7·5 tons per square inch, equal to a factor of safety of 4. Ends of girders simply supported.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND GIRDERS.

Composition and Properties.



Composed of		Weight per foot in lbs.	Area in square inches.	Maxi- mum Moment of Inertia. X-X	Maxi- mum Modulus of Section. X-X	Safe Distributed Load on 1 foot Span for		Deflection Coefficient. X-X
Two Steel Channels.	Plates, each Flange to form.					Girder.	1 in. Plate width.	
15 x 4	16 x 1/2	168	48.7	2039	247.2	1236	56.4	.001136
"	" x 1/2	154 1/2	44.7	1798	221.3	1106	47.0	.001151
"	" x 1/2	141	40.7	1565	195.6	978	37.5	.001172
"	12 x 1/2	147 1/2	42.7	1667	202.0	1010	56.4	.001136
"	" x 1/2	137 1/2	39.7	1493	183.8	919	47.0	.001151
"	" x 1/2	127	36.7	1324	165.6	828	37.5	.001172
12 x 3 1/2	16 x 1/2	150	43.3	1227	181.8	909	45.2	.001389
"	" x 1/2	136 1/2	39.3	1066	160.9	804	37.6	.001415
"	" x 1/2	122 1/2	35.3	912	140.2	701	30.0	.001442
"	12 x 1/2	119 1/2	34.3	867	130.9	654	37.6	.001415
"	" x 1/2	109	31.3	755	116.2	581	30.0	.001442
10 x 3 1/2	16 x 1/2	140 1/2	40.6	839	145.9	729	37.7	.001630
"	" x 1/2	127	36.6	723	128.5	642	31.4	.001667
"	" x 1/2	113 1/2	32.6	612	111.2	556	25.0	.001704
"	12 x 1/2	110	31.6	582	103.4	517	31.4	.001667
"	" x 1/2	100	28.6	501	91.1	455	25.0	.001704
9 x 3 1/2	16 x 1/2	135	38.9	674	128.3	641	34.0	.001785
"	" x 1/2	121 1/2	34.9	577	112.6	563	28.3	.001829
"	" x 1/2	107 1/2	30.9	485	96.9	484	22.6	.001875
"	12 x 1/2	104 1/2	29.9	461	89.9	449	28.3	.001829
"	" x 1/2	94	26.9	394	78.9	394	22.6	.001875
8 x 3 1/2	16 x 1/2	116	33.4	450	97.4	487	25.1	.002027
"	" x 1/2	102 1/2	29.4	376	83.5	417	20.1	.002083
"	12 x 1/2	99	28.4	357	77.2	386	25.1	.002027
"	" x 1/2	88 1/2	25.4	303	67.4	337	20.1	.002083
7 x 3 1/2	16 x 1/2	111	31.9	342	83.0	415	22.0	.002273
"	" x 1/2	97 1/2	27.9	283	70.7	353	17.6	.002344
"	12 x 1/2	94	26.9	269	65.3	326	22.0	.002273
"	" x 1/2	84	23.9	227	56.6	283	17.6	.002344

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2 1/2 per cent. over this must be allowed. See page 7.

Let  $\delta$  = deflection, K = deflection coefficient, and L = span in feet, then  $\delta = K \times L^4$ .

For full explanations of tables, see notes commencing on page 108.

For formulae, explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.



## PLATE GIRDERS.

Safe Distributed Loads, in Tons.

Reference Mark.	Size, D x B inches.	SPANS IN FEET.															
		14	16	18	20	22	24	26	28	30	32	34	36	40	44		
41 F	40 x 12 $\frac{1}{2}$	145	127	113	102	92	85	78	72	68	63	60	56	51	46	3	3
40 F	" x 10 $\frac{1}{2}$	122	107	95	85	77	71	65	61	57	53	50	47	42	38	9	9
31 F	36 x 12 $\frac{1}{2}$	127	111	98	88	80	74	68	63	59	55	52	49	44	40	4	4
30 F	" x 10 $\frac{1}{2}$	106	93	82	74	67	62	57	53	49	46	43	41	37	33	8	8
21 F	32 x 10 $\frac{1}{2}$	91	79	70	63	58	53	49	45	42	39	37	35	31	29	0	0
20 F	" x 8 $\frac{1}{2}$	80	70	62	56	51	46	43	40	37	35	33	31	28	25	5	5
11 F	28 x 10 $\frac{1}{2}$	76	67	59	53	48	44	41	38	35	33	31	29	26	24	4	4
10 F	" x 8 $\frac{1}{2}$	67	58	52	47	42	39	36	33	31	29	27	26	23	21	4	4
35 G	42 x 12	262	229	204	183	167	153	141	131	122	114	108	102	91	83	4	4
34 G	41 $\frac{1}{2}$ x "	244	214	190	171	155	142	131	122	114	107	100	95	85	77	7	7
33 G	41 $\frac{1}{2}$ x "	226	198	176	158	144	132	122	113	105	99	93	88	79	72	1	1
32 G	41 $\frac{1}{2}$ x "	208	182	162	146	133	122	112	104	97	91	85	81	73	66	4	4
31 G	41 x "	191	167	148	133	121	111	103	95	89	83	78	74	66	60	7	7
25 G	38 x 12	231	202	180	162	147	135	125	116	108	101	95	90	81	73	6	6
24 G	37 $\frac{1}{2}$ x "	215	188	167	151	137	126	116	108	100	94	88	83	75	68	5	5
23 G	37 $\frac{1}{2}$ x "	199	174	155	139	127	116	107	99	93	87	82	77	69	63	4	4
22 G	37 $\frac{1}{2}$ x "	183	160	142	128	116	107	98	91	85	80	75	71	64	58	3	3
21 G	37 x "	167	146	130	117	106	97	90	83	77	72	68	65	58	53	2	2
15 G	34 x 12	193	169	150	135	123	112	104	96	90	84	79	75	67	61	4	4
14 G	33 $\frac{1}{2}$ x "	178	156	139	125	113	104	96	89	83	78	73	69	62	56	8	8
13 G	33 $\frac{1}{2}$ x "	164	144	128	115	104	95	88	82	76	71	67	63	57	52	3	3
12 G	33 $\frac{1}{2}$ x "	150	131	117	105	95	87	80	75	70	65	61	58	52	47	7	7
11 G	33 x "	136	119	105	95	86	79	73	67	63	59	55	52	47	43	2	2
5 G	30 x 12	165	144	128	115	105	96	88	82	77	72	68	64	57	52	5	5
4 G	29 $\frac{1}{2}$ x "	152	133	118	107	97	89	82	76	71	66	62	59	53	48	5	5
3 G	29 $\frac{1}{2}$ x "	140	122	109	98	89	81	75	70	65	61	57	54	49	44	5	5
2 G	29 $\frac{1}{2}$ x "	127	111	99	89	81	74	68	63	59	55	52	49	44	40	6	6
1 G	29 x "	115	101	89	80	73	67	62	57	53	50	47	44	40	36	6	6

The above safe loads are based on the following assumptions :-

Safe working stress = 7.5 tons per square inch, equal to a factor of safety of 4. Ends of girders simply supported. Requisite rivet pitch adopted. Webs adequately stiffened. Efficient lateral support provided.

Weights per foot are for sections of girders only; they do not include any allowance for stiffeners.

# REDPATH, BROWN & CO., LIMITED.

## PLATE GIRDERS.

Composition and Properties.



Composed of			Weight per foot in lbs.	Area in square inches.	Maxi- mum Moment of Inertia. X-X	Maxi- mum Modu- lus of Section X-X	Safe Distributed Load on 1 foot Span for		Deflection Coefficient. X-X
Web Plate.	Each Flange.						Girder.	1 in. Plate Width.	
	Plate Thick- ness.	Section of Angles.							
40 × <small>CSA</small>		6 × 4 × $\frac{1}{2}$	118	34.0	8159	408.0	2040		.000469
" <small>CSA</small>		5 × 3 × $\frac{1}{2}$	104½	30.0	6844	342.2	1711		.000469
36 × <small>CSA</small>		6 × 4 × $\frac{1}{2}$	113	32.5	6401	355.6	1778		.000521
" <small>CSA</small>		5 × 3 × $\frac{1}{2}$	99½	28.5	5359	297.7	1488		.000521
32 × <small>CSA</small>		5 × 3 × $\frac{1}{2}$	94½	27.0	4085	255.3	1276		.000586
" <small>CSA</small>		4 × 3 × $\frac{1}{2}$	87½	25.0	3596	224.7	1123		.000586
28 × <small>CSA</small>		5 × 3 × $\frac{1}{2}$	89	25.5	3007	214.8	1074		.000670
" <small>CSA</small>		4 × 3 × $\frac{1}{2}$	82½	23.5	2636	188.3	941		.000670
40 × <small>CSA</small>	1 $\frac{1}{2}$	4 × 4 × $\frac{1}{2}$	190½	54.0	15427	734.6	3673	200.1	.000447
" <small>CSA</small>	$\frac{3}{4}$	"	180½	51.0	14290	684.5	3422	175.1	.000450
" <small>CSA</small>	$\frac{1}{2}$	"	170	48.0	13166	634.5	3172	150.0	.000452
" <small>CSA</small>	$\frac{1}{4}$	"	160	45.0	12056	584.5	2922	125.0	.000455
" <small>CSA</small>	$\frac{1}{8}$	"	150	42.0	10960	534.6	2673	100.0	.000457
36 × <small>CSA</small>	1 $\frac{1}{2}$	4 × 4 × $\frac{1}{2}$	185½	52.5	12319	648.3	3241	180.1	.000493
" <small>CSA</small>	$\frac{3}{4}$	"	175½	49.5	11389	603.3	3016	157.6	.000497
" <small>CSA</small>	$\frac{1}{2}$	"	165	46.5	10471	558.3	2791	135.0	.000500
" <small>CSA</small>	$\frac{1}{4}$	"	155	43.5	9565	513.5	2567	112.5	.000503
" <small>CSA</small>	$\frac{1}{8}$	"	145	40.5	8672	468.7	2343	90.0	.000507
32 × <small>CSA</small>	1 $\frac{1}{2}$	3½ × 3½ × $\frac{1}{2}$	173½	49.0	9191	540.6	2703	160.2	.000552
" <small>CSA</small>	$\frac{3}{4}$	"	163½	46.0	8447	500.5	2502	140.1	.000556
" <small>CSA</small>	$\frac{1}{2}$	"	153	43.0	7714	460.5	2302	120.1	.000560
" <small>CSA</small>	$\frac{1}{4}$	"	143	40.0	6991	420.5	2102	100.0	.000564
" <small>CSA</small>	$\frac{1}{8}$	"	133	37.0	6280	380.6	1903	80.0	.000568
28 × <small>CSA</small>	1 $\frac{1}{2}$	3½ × 3½ × $\frac{1}{2}$	168½	47.5	6936	462.4	2312	140.2	.000625
" <small>CSA</small>	$\frac{3}{4}$	"	158½	44.5	6357	427.3	2136	122.6	.000630
" <small>CSA</small>	$\frac{1}{2}$	"	148	41.5	5789	392.4	1962	105.1	.000636
" <small>CSA</small>	$\frac{1}{4}$	"	138	38.5	5229	357.5	1787	87.5	.000641
" <small>CSA</small>	$\frac{1}{8}$	"	128	35.5	4679	322.7	1613	70.0	.000647

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2 1/2 per cent. over this must be allowed. See page 7.

When "plate thickness, each flange," exceeds 1/2 of an inch, two plates may be used.

Let  $\delta$  = deflection,  $K$  = deflection coefficient, and  $L$  = span in feet, then  $\delta = K \times L^3$ .

For full explanations of tables, see notes commencing page 108.

For formulae, explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.



## BOX PLATE GIRDERS.

Safe Distributed Loads, in Tons.

Reference Mark.	Size, D x B inches.	SPANS IN FEET.													
		14	16	18	20	22	24	26	28	30	32	34	36	40	44
37H	42½ x 18	439	384	341	307	279	256	236	219	205	192	180	170	153	139
35H	42 x "	382	334	297	267	243	223	205	191	178	167	157	148	133	121
34H	41½ x "	354	309	275	247	225	206	190	177	165	155	145	137	124	112
33H	41½ x "	325	285	253	227	207	190	175	162	152	142	134	126	114	103
32H	41½ x "	297	260	231	208	189	173	160	148	138	130	122	115	104	94.4
31H	41 x "	268	235	209	188	171	156	144	134	125	117	110	104	94.0	85.4
27H	38½ x 18	387	339	301	271	246	226	208	193	180	169	159	150	135	123
25H	38 x "	336	294	261	235	214	196	181	168	157	147	138	130	117	107
24H	37½ x "	311	272	242	217	197	181	167	155	145	136	128	120	108	98.8
23H	37½ x "	285	249	222	199	181	166	153	142	133	124	117	111	100	90.7
22H	37½ x "	259	227	202	181	165	151	140	130	121	113	107	101	90.8	82.6
21H	37 x "	234	205	182	163	149	136	126	117	109	102	96.4	91.0	81.9	74.4
17H	34½ x 18	329	288	256	230	209	192	177	164	153	144	135	128	115	104
15H	34 x "	283	248	220	198	180	165	152	141	132	124	116	110	99.1	90.1
14H	33½ x "	260	228	202	182	165	152	140	130	121	114	107	101	91.1	82.8
13H	33½ x "	238	208	185	166	151	138	128	119	111	104	97.9	92.4	83.2	75.6
12H	33½ x "	215	188	167	150	137	125	116	107	100	94.1	88.5	83.6	75.2	68.4
11H	33 x "	192	168	149	134	122	112	103	96.2	89.7	84.1	79.2	74.8	67.3	61.2
7H	30½ x 18	281	246	219	197	179	164	151	140	131	123	116	109	98.5	89.5
5H	30 x "	241	211	188	169	153	141	130	121	112	105	99.4	93.9	84.5	76.8
4H	29½ x "	221	194	172	155	141	129	119	111	103	97.0	91.3	86.2	77.6	70.5
3H	29½ x "	202	176	157	141	128	117	108	101	94.1	88.3	83.1	78.4	70.6	64.2
2H	29½ x "	182	159	141	127	116	106	97.9	91.0	84.9	79.6	74.9	70.7	63.6	57.9
1H	29 x "	162	142	126	113	103	94.5	87.3	81.0	75.6	70.9	66.7	63.0	56.7	51.5

The above safe loads are based on the following assumptions:—

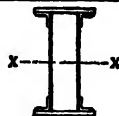
Safe working stress = 7.5 tons per square inch, equal to a factor of safety of 4. Ends of girders simply supported. Requisite rivet pitch adopted. Webs adequately stiffened. Efficient lateral support provided.

Weights per foot are for sections of girders only; they do not include any allowance for stiffeners.

# REDPATH, BROWN & CO., LIMITED.

## BOX PLATE GIRDERS.

Composition and Properties.



Composed of			Weight per foot in lbs.	Area in square inches.	Maxi- mum Moment of Inertia. X-X	Maxi- mum Modu- lus of Section X-X	Safe Distributed Load on 1 foot Span for		Deflection Coefficient. X-X
Two Web Plates	Each Flange.						Girder.	1 in. Plate width.	
	Plate Thick- ness.	Section of Angles.							
40 x $\frac{3}{8}$	1 $\frac{1}{2}$	4 x 4 x $\frac{1}{2}$	315	90.0	26126	1229.0	6145	250.3	.000441
"	1	"	284 $\frac{1}{2}$	81.0	22472	1070.0	5350	200.1	.000447
"	$\frac{7}{8}$	"	269 $\frac{1}{2}$	76.5	20677	990.5	4952	175.1	.000450
"	$\frac{3}{4}$	"	254	72.0	18904	911.1	4555	150.0	.000452
"	$\frac{5}{8}$	"	238	67.5	17152	831.6	4158	125.0	.000455
"	$\frac{1}{2}$	"	223 $\frac{1}{2}$	63.0	15421	752.1	3760	100.0	.000458
36 x $\frac{3}{8}$	1 $\frac{1}{2}$	4 x 4 x $\frac{1}{2}$	305	87.0	20880	1084.0	5420	225.3	.000487
"	1	"	274 $\frac{1}{2}$	78.0	17885	941.3	4706	180.1	.000494
"	$\frac{7}{8}$	"	259	73.5	16417	869.9	4249	157.6	.000497
"	$\frac{3}{4}$	"	244	69.0	14968	798.3	3991	135.0	.000500
"	$\frac{5}{8}$	"	228 $\frac{1}{2}$	64.5	13539	727.0	3635	112.5	.000504
"	$\frac{1}{2}$	"	213	60.0	12128	655.5	3277	90.0	.000507
32 x $\frac{3}{8}$	1 $\frac{1}{2}$	3 $\frac{1}{2}$ x 3 $\frac{1}{2}$ x $\frac{1}{2}$	298	82.0	15884	920.8	4604	200.3	.000544
"	1	"	257 $\frac{1}{2}$	73.0	13483	793.1	3965	160.2	.000552
"	$\frac{7}{8}$	"	242	68.5	12309	729.4	3647	140.1	.000556
"	$\frac{3}{4}$	"	227	64.0	11151	665.7	3328	120.1	.000560
"	$\frac{5}{8}$	"	211 $\frac{1}{2}$	59.5	10011	602.2	3011	100.0	.000564
"	$\frac{1}{2}$	"	196	55.0	8888	538.7	2693	80.0	.000569
28 x $\frac{3}{8}$	1 $\frac{1}{2}$	3 $\frac{1}{2}$ x 3 $\frac{1}{2}$ x $\frac{1}{2}$	278	79.0	12020	788.1	3940	175.4	.000615
"	1	"	247	70.0	10146	676.4	3382	140.2	.000625
"	$\frac{7}{8}$	"	232	65.5	9233	620.7	3103	122.6	.000631
"	$\frac{3}{4}$	"	216 $\frac{1}{2}$	61.0	8335	565.0	2825	105.0	.000636
"	$\frac{5}{8}$	"	201 $\frac{1}{2}$	56.5	7452	509.5	2547	87.5	.000641
"	$\frac{1}{2}$	"	186	52.0	6583	454.0	2270	70.0	.000647

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2 1/2 per cent. over this must be allowed. See page 7.

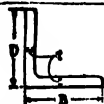
When "plate thickness, each flange," exceeds 1/2 of an inch, two plates may be used.

Let  $\delta$  = deflection, K = deflection coefficient, and L = span in feet, then  $\delta = K \times L^2$ .

For full explanations of tables, see notes commencing page 108.

For formulas, explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.



## STEEL EQUAL ANGLES.

Distributed **BREAKING** Loads, in Tons.

Reference Mark.	Size, D x B x t inches.	SPANS IN FEET.															
		3	4	5	6	7	8	9	10	11	12	13	14	15	16		
BSEA 14g	6 x 6 x $\frac{3}{8}$	43.7	32.7	26.2	21.8	18.7	16.4	14.5	13.1	11.9	10.9	10.1	9.3	8.7	8.2		
BSEA 14f	" x $\frac{5}{8}$	36.9	27.7	22.2	18.5	15.8	13.8	12.3	11.1	10.1	9.2	8.5	7.9	7.4	6.9		
BSEA 14e	" x $\frac{1}{2}$	30.0	22.5	18.0	15.0	12.8	11.2	10.0	9.0	8.1	7.5	6.9	6.4	6.0	5.6		
BSEA 13g	5 x 5 x $\frac{3}{8}$	29.6	22.2	17.8	14.8	12.7	11.1	9.8	8.9	8.1	7.4	6.8	6.3	5.9	5.5		
BSEA 13f	" x $\frac{5}{8}$	25.2	18.9	15.1	12.6	10.8	9.4	8.4	7.5	6.8	6.3	5.8	5.4	5.0	4.7		
BSEA 13e	" x $\frac{1}{2}$	20.5	15.3	12.3	10.2	8.8	7.7	6.8	6.1	5.6	5.1	4.7	4.4	4.1	3.8		
BSEA 12g	4 $\frac{1}{2}$ x 4 $\frac{1}{2}$ x $\frac{3}{8}$	23.7	17.8	14.2	11.8	10.1	8.9	7.9	7.1	6.4	5.9	5.4	5.0	4.7	4.4		
BSEA 12f	" x $\frac{5}{8}$	20.1	15.1	12.1	10.1	8.6	7.5	6.7	6.0	5.5	5.0	4.6	4.3	4.0	3.7		
BSEA 12e	" x $\frac{1}{2}$	16.4	12.3	9.8	8.2	7.0	6.1	5.5	4.9	4.5	4.1	3.8	3.5	3.3	3.0		
BSEA 11g	4 x 4 x $\frac{3}{8}$	18.4	13.8	11.1	9.2	7.9	6.9	6.1	5.5	5.0	4.6	4.2	3.9	3.7	3.4		
BSEA 11f	" x $\frac{5}{8}$	15.7	11.8	9.4	7.8	6.7	5.9	5.2	4.7	4.3	3.9	3.6	3.3	3.1	2.9		
BSEA 11e	" x $\frac{1}{2}$	12.8	9.6	7.7	6.4	5.5	4.8	4.3	3.8	3.5	3.2	2.9	2.7	2.5	2.4		
BSEA 11d	" x $\frac{3}{8}$	9.8	7.4	5.9	4.9	4.2	3.7	3.3	2.9	2.7	2.4	2.2	2.1	1.9	1.8		
BSEA 10f	3 $\frac{1}{2}$ x 3 $\frac{1}{2}$ x $\frac{5}{8}$	11.8	8.8	7.1	5.9	5.0	4.4	3.9	3.5	3.2	2.9	2.7	2.5	2.3	2.2		
BSEA 10e	" x $\frac{1}{2}$	9.7	7.3	5.8	4.8	4.1	3.6	3.2	2.9	2.6	2.4	2.2	2.0	1.9	1.8		
BSEA 10d	" x $\frac{3}{8}$	7.4	5.6	4.4	3.7	3.2	2.8	2.5	2.2	2.0	1.8	1.7	1.6	1.5	1.4		

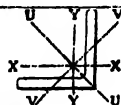
NOTE PARTICULARLY that BREAKING load values are given in this table, based on an ultimate stress of 30 tons per square inch, and corresponding to axis XX or YY.

Angles as purlins or side framing bars (usually bolted at each end and continuous over two or more spans) may be stressed safely up to 10 tons per square inch, equal to a factor of safety of 3. For angles as beams over single spans, with ends simply supported, the factor of safety should be 4.

# REDPATH, BROWN & CO., LIMITED.

## STEEL EQUAL ANGLES.

### Dimensions and Properties.



Size, D x B x t inches.	Weight per foot in lbs.	Area in square inches.	Moments of Inertia.			Modulus of Section.	Distri- buted Breaking Load on 1-ft. Span.	Deflection Coefficient.
			Axis XX or Axis YY	Axis UU Max.	Axis VV Min.			
6 x 6 x 3/4	28.70	8.441	27.79	44.03	11.55	6.55	131.1	.008867
" x 5/8	24.18	7.113	23.78	37.66	9.90	5.54	110.9	.008748
" x 1/2	19.55	5.753	19.52	31.03	8.01	4.50	90.0	.008647
5 x 5 x 3/4	23.59	6.938	15.53	24.67	6.39	4.45	89.0	.010748
" x 5/8	19.92	5.860	13.36	21.09	5.63	3.78	75.5	.010606
" x 1/2	16.15	4.751	11.02	17.48	4.56	3.07	61.5	.010464
4 1/2 x 4 1/2 x 3/4	21.05	6.189	11.08	17.69	4.47	3.56	71.1	.012047
" x 5/8	17.80	5.236	9.56	15.16	3.96	3.03	60.5	.011871
" x 1/2	14.46	4.252	7.92	12.62	3.22	2.47	49.4	.011697
4 x 4 x 3/4	18.49	5.437	7.57	12.01	3.14	2.77	55.3	.013702
" x 5/8	15.66	4.609	6.56	10.40	2.73	2.36	47.2	.013480
" x 1/2	12.75	3.750	5.46	8.70	2.22	1.93	38.6	.013256
" x 5/16	9.72	2.859	4.26	6.78	1.74	1.48	29.5	.013030
3 1/2 x 3 1/2 x 5/8	13.55	3.985	4.27	6.70	1.84	1.77	35.5	.015580
" x 1/2	11.05	3.251	3.57	5.65	1.50	1.46	29.1	.015288
" x 5/16	8.45	2.485	2.80	4.45	1.15	1.12	22.4	.014904

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2 1/2 per cent. over this must be allowed. See page 7.

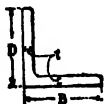
All above sections are in our stocks.

For full explanations of tables, see notes commencing page 108.

For formulas, explanations of properties, &c., see Part IV.

Let  $\delta$  = deflection in inches,  $K$  = deflection coefficient,  $L$  = span in feet, and  $F$  = factor of safety, then  $\delta = \frac{K \times L^3}{F}$

# REDPATH, BROWN & CO., LIMITED.



## STEEL EQUAL ANGLES.

Distributed BREAKING Loads, in Tons.

Reference Mark.	Size, D × B × t inches.	SPANS IN FEET.														
		2	3	4	5	6	7	8	9	10	11	12	13	14	15	
BSEA 9f	3 × 3 × $\frac{5}{8}$	12.7	8.5	6.3	5.1	4.2	3.6	3.2	2.8	2.5	2.3	2.1	1.9	1.8	1.7	
BSEA 9e	" × $\frac{1}{2}$	10.5	7.0	5.2	4.2	3.5	3.0	2.6	2.3	2.1	1.9	1.7	1.6	1.5	1.4	
BSEA 9d	" × $\frac{3}{8}$	8.1	5.4	4.0	3.2	2.7	2.3	2.0	1.8	1.6	1.4	1.3	1.2	1.1	1.0	
BSEA 9c	" × $\frac{1}{4}$	6.8	4.5	3.4	2.7	2.2	1.9	1.7	1.5	1.3	1.2	1.1	1.0			
BSEA 9b	" × $\frac{1}{8}$	5.5	3.6	2.7	2.2	1.8	1.5	1.3	1.2	1.1	1.0					
BSEA 7e	2½ × 2½ × $\frac{1}{2}$	7.0	4.7	3.5	2.8	2.3	2.0	1.7	1.5	1.4	1.2	1.1				
BSEA 7d	" × $\frac{3}{8}$	5.4	3.6	2.7	2.2	1.8	1.5	1.3	1.2	1.1						
BSEA 7c	" × $\frac{1}{4}$	4.6	3.1	2.3	1.8	1.5	1.3	1.1	1.0	0.9						
BSEA 7b	" × $\frac{1}{8}$	3.7	2.5	1.8	1.5	1.2	1.0	0.9	0.8	0.7						
BSEA 6c	2½ × 2½ × $\frac{3}{8}$	3.7	2.5	1.8	1.5	1.2	1.0	0.9	0.8	0.7						
BSEA 6b	" × $\frac{1}{4}$	3.0	2.0	1.5	1.2	1.0	0.8	0.7	0.6	0.6						
BSEA 5b	2 × 2 × $\frac{1}{4}$	2.3	1.5	1.1	0.9	0.7	0.6	0.6	0.5	0.4						
BSEA 5a	" × $\frac{3}{8}$	1.8	1.2	0.9	0.7	0.6	0.5	0.4	0.4	0.3						

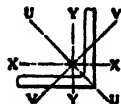
NOTE PARTICULARLY that BREAKING load values are given in this table, based on an ultimate stress of 30 tons per square inch, and corresponding to axis XX or YY.

Angles as purlins or side framing bars (usually bolted at each end and continuous over two or more spans) may be stressed safely up to 10 tons per square inch, equal to a factor of safety of 3. For angles as beams over single spans, with ends simply supported, the factor of safety should be 4.

# REDPATH, BROWN & CO., LIMITED.

## STEEL EQUAL ANGLES.

Dimensions and Properties.



Size, D x B x t inches.	Weight per foot in lbs.	Area in square inches.	Moments of Inertia.			Modulus of Section.	Distributed Breaking Load on 1-ft. Span.	Deflection Coefficient.
			Axis XX or Axis YY.	Axis UU Max.	Axis VV Min.	Axis XX or Axis YY.	Axis XX or Axis YY.	Axis XX or Axis YY.
3 x 3 x $\frac{1}{4}$	11.43	3.362	2.59	4.05	1.13	1.27	25.5	.018455
" x $\frac{1}{2}$	9.36	2.753	2.18	3.44	0.92	1.05	21.0	.018063
" x $\frac{3}{8}$	7.18	2.112	1.72	2.73	0.71	0.81	16.2	.017664
" x $\frac{1}{8}$	6.05	1.779	1.47	2.34	0.60	0.68	13.7	.017467
" x $\frac{1}{4}$	4.90	1.440	1.21	1.91	0.50	0.55	11.1	.017258
2½ x 2½ x $\frac{1}{4}$	7.65	2.250	1.20	1.88	0.52	0.71	14.1	.022034
" x $\frac{3}{8}$	5.89	1.734	0.96	1.52	0.40	0.55	10.9	.021454
" x $\frac{1}{8}$	4.98	1.464	0.82	1.31	0.33	0.46	9.2	.021175
" x $\frac{1}{4}$	4.04	1.187	0.68	1.08	0.27	0.37	7.5	.020842
2½ x 2½ x $\frac{3}{16}$	4.45	1.310	0.59	0.94	0.24	0.37	7.5	.023690
" x $\frac{1}{4}$	3.61	1.063	0.49	0.77	0.20	0.30	6.0	.023336
2 x 2 x $\frac{1}{4}$	3.19	0.938	0.33	0.52	0.14	0.23	4.7	.026428
" x $\frac{3}{16}$	2.43	0.715	0.26	0.41	0.11	0.18	3.6	.025931

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

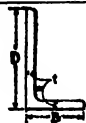
All above sections are in our stocks.

For full explanations of tables, see notes commencing page 108.

For formulae, explanations of properties, &c., see Part IV.

Let  $\delta$  = deflection in inches, K = deflection coefficient, L = span in feet, and F = factor of safety, then  $\delta = \frac{K \times L^3}{F}$

# REDPATH, BROWN & CO., LIMITED.



## STEEL UNEQUAL ANGLES.

Long Leg Vertical.

Distributed **BREAKING** Loads, in Tons.

Reference Mark.	Size, D x B x t inches.	SPANS IN FEET.															
		3	4	5	6	7	8	9	10	11	12	13	14	15	16		
BSUA 25g	7 x 3½ x ½	54.0	40.5	32.4	27.0	23.1	20.2	18.0	16.2	14.7	13.5	12.4	11.6	10.8	10.1		
BSUA 25f	" x ½	45.7	34.3	27.4	22.8	19.6	17.1	15.2	13.7	12.4	11.4	10.5	9.8	9.1	8.5		
BSUA 25e	" x ½	37.2	27.9	22.3	18.6	15.9	13.9	12.4	11.1	10.1	9.3	8.5	7.9	7.4	6.9		
BSUA 21f	6 x 4 x ½	34.8	26.1	20.9	17.4	14.9	13.0	11.6	10.4	9.5	8.7	8.0	7.4	6.9	6.5		
BSUA 21e	" x ½	28.3	21.2	16.9	14.1	12.1	10.6	9.4	8.5	7.7	7.0	6.5	6.0	5.6	5.3		
BSUA 20f	6 x 3½ x ½	34.1	25.5	20.4	17.0	14.6	12.8	11.3	10.2	9.3	8.5	7.8	7.3	6.8	6.4		
BSUA 20e	" x ½	27.7	20.8	16.6	13.8	11.8	10.4	9.2	8.3	7.5	6.9	6.4	5.9	5.5	5.2		
BSUA 20d	" x ½	21.0	15.8	12.6	10.5	9.0	7.8	7.0	6.3	5.7	5.2	4.8	4.5	4.2	3.9		
RBUA 63f	6 x 3 x ½	33.1	24.8	19.8	16.5	14.2	12.4	11.0	9.9	9.0	8.2	7.6	7.1	6.6	6.2		
RBUA 63e	" x ½	27.0	20.2	16.2	13.5	11.5	10.1	9.0	8.1	7.3	6.7	6.2	5.7	5.4	5.0		
RBUA 63d	" x ½	20.6	15.4	12.3	10.3	8.8	7.7	6.8	6.1	5.6	5.1	4.7	4.4	4.1	3.8		

NOTE PARTICULARLY that BREAKING load values are given in this table, based on an ultimate stress of 30 tons per square inch, and corresponding to axis XX.

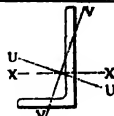
Angles as purlins or side framing bars (usually bolted at each end and continuous over two or more spans) may be stressed safely up to 10 tons per square inch, equal to a factor of safety of 3. For angles as beams over single spans, with ends simply supported, the factor of safety should be 4.

# REDPATH, BROWN & CO., LIMITED.

## STEEL UNEQUAL ANGLES.

Long Leg Vertical.

Dimensions and Properties.



Size, D x B x t inches.	Weight per foot in lbs.	Area in square inches.	Moments of Inertia.			Modulus of Section.	Distrib- uted Breaking Load on 1-ft. Span.	Deflection Coefficient.
			Axis XX.	Axis UU Max.	Axis VV Min.			
7 x 3½ x ¾	24.86	7.313	35.68	37.73	3.90	8.11	162.2	.008523
" x ¾	20.98	6.172	30.55	32.32	3.38	6.86	137.2	.008427
" x ½	17.00	5.000	25.10	26.64	2.74	5.58	111.6	.008334
6 x 4 x ¾	19.92	5.860	20.80	23.83	4.33	5.22	104.4	.009411
" x ½	16.15	4.750	17.13	19.72	3.51	4.24	84.9	.009294
6 x 3½ x ¾	18.87	5.550	19.89	21.77	3.09	5.11	102.3	.009648
" x ½	15.31	4.502	16.39	18.00	2.53	4.16	83.2	.009525
" x ¾	11.64	3.424	12.59	13.83	1.98	3.15	63.1	.009399
6 x 3 x ¾	17.80	5.230	18.79	19.84	2.08	4.97	99.4	.009921
" x ½	14.46	4.252	15.50	16.44	1.68	4.05	81.0	.009792
" x ¾	11.00	3.236	12.00	12.72	1.33	3.09	61.8	.009665

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

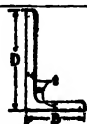
All above sections are in our stocks.

For full explanations of tables, see notes commencing page 108.

For formulae, explanations of properties, &c., see Part IV.

Let  $\delta$  = deflection in inches,  $K$  = deflection coefficient,  $L$  = span in feet, and  $F$  = factor of safety, then  $\delta = \frac{K \times L^3}{F}$

# REDPATH, BROWN & CO., LIMITED.



## STEEL UNEQUAL ANGLES.

Long Leg Vertical.

Distributed **BREAKING** Loads, in Tons.

Reference Mark.	Size, D x B x t inches.	SPANS IN FEET.														
		2	3	4	5	6	7	8	9	10	11	12	13	14	15	
BSUA 17 <i>f</i>	5 x 4 x $\frac{5}{8}$	36.6	24.4	18.3	14.6	12.2	10.4	9.1	8.1	7.3	6.6	6.1	5.6	5.2	4.8	
BSUA 17 <i>e</i>	" x $\frac{1}{2}$	29.9	19.9	14.9	11.9	9.9	8.5	7.4	6.6	5.9	5.4	4.9	4.6	4.2	3.9	
BSUA 17 <i>d</i>	" x $\frac{3}{8}$	22.8	15.2	11.4	9.1	7.6	6.5	5.7	5.0	4.5	4.1	3.8	3.5	3.2	3.0	
BSUA 15 <i>f</i>	5 x 3. x $\frac{5}{8}$	35.0	23.3	17.5	14.0	11.6	10.0	8.7	7.7	7.0	6.3	5.8	5.3	5.0	4.6	
BSUA 15 <i>e</i>	" x $\frac{1}{2}$	28.6	19.0	14.2	11.4	9.5	8.1	7.1	6.3	5.7	5.2	4.7	4.4	4.0	3.8	
BSUA 15 <i>d</i>	" x $\frac{3}{8}$	21.8	14.5	10.9	8.7	7.2	6.2	5.4	4.8	4.3	3.9	3.6	3.3	3.1	2.9	
BSUA 11 <i>e</i>	4 x 3 x $\frac{1}{2}$	18.5	12.3	9.2	7.4	6.1	5.3	4.6	4.1	3.7	3.3	3.1	2.8	2.6	2.4	
BSUA 11 <i>d</i>	" x $\frac{3}{8}$	14.2	9.5	7.1	5.7	4.7	4.0	3.5	3.1	2.8	2.6	2.4	2.2	2.0	1.9	
BSUA 7 <i>d</i>	3 x 2 $\frac{1}{2}$ x $\frac{3}{8}$	7.9	5.2	3.9	3.1	2.6	2.2	1.9	1.7	1.5	1.4	1.3	1.2	1.1	1.0	
BSUA 7 <i>c</i>	" x $\frac{1}{4}$	6.6	4.4	3.3	2.6	2.2	1.9	1.6	1.4	1.3	1.2	1.1	1.0	0.9	0.8	

NOTE PARTICULARLY that **BREAKING** load values are given in this table, based on an ultimate stress of 30 tons per square inch, and corresponding to axis xx.

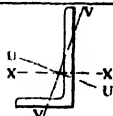
Angles as purlins or side framing bars (usually bolted at each end and continuous over two or more spans) may be stressed safely up to 10 tons per square inch, equal to a factor of safety of 3. For angles as beams over single spans, with ends simply supported, the factor of safety should be 4.

# REDPATH, BROWN & CO., LIMITED.

## STEEL UNEQUAL ANGLES.

Long Leg Vertical.

Dimensions and Properties.



Size, D x B x t inches.	Weight per foot in lbs.	Area in square inches.	Moments of Inertia.			Modulus of Section.	Distrib- uted Breaking Load on 1-ft. Span.	Deflection Coefficient.
			Axis XX.	Axis UU Max.	Axis VV Min.			
5 x 4 x $\frac{3}{8}$	17.80	5.236	12.44	15.84	3.61	3.66	73.2	.011046
" x $\frac{1}{2}$	14.46	4.252	10.29	13.12	3.00	2.99	59.8	.010892
" x $\frac{5}{8}$	11.00	3.236	7.96	10.15	2.34	2.28	45.6	.010748
5 x 3 x $\frac{3}{8}$	15.67	4.609	11.26	12.38	1.88	3.50	70.0	.016569
" x $\frac{1}{2}$	12.75	3.749	9.33	10.30	1.54	2.86	57.1	.011486
" x $\frac{5}{8}$	9.72	2.859	7.24	8.00	1.21	2.18	43.7	.011309
4 x 3 x $\frac{1}{2}$	11.05	3.251	4.98	6.06	1.29	1.85	37.1	.013967
" x $\frac{5}{8}$	8.45	2.485	3.89	4.74	1.02	1.42	28.4	.013717
3 x 2 $\frac{1}{2}$ x $\frac{3}{8}$	6.53	1.921	1.62	2.12	0.52	0.79	15.8	.018249
" x $\frac{1}{2}$	5.51	1.620	1.39	1.82	0.44	0.67	13.3	.018033

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of  $2\frac{1}{2}$  per cent. over this must be allowed. See page 7.

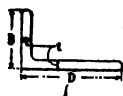
All above sections are in our stocks.

For full explanations of tables, see notes commencing page 108.

For formulae, explanations of properties, &c., see Part IV.

Let  $\delta$  = deflection in inches,  $K$  = deflection coefficient,  $L$  = span in feet, and  $F$  = factor of safety, then  $\delta = \frac{K \times L^3}{F}$

# REDPATH, BROWN & CO., LIMITED.



## STEEL UNEQUAL ANGLES.

Short Leg Vertical.

Distributed **BREAKING** Loads, in Tons.

Reference Mark.	Size, D x B x t inches.	SPANS IN FEET.															
		3	4	5	6	7	8	9	10	11	12	13	14	15	16		
BSUA 25g	7 x 3½ x ½	15.0	11.3	9.0	7.5	6.4	5.6	5.0	4.5	4.1	3.7	3.4	3.2	3.0	2.8		
BSUA 25f	" x ½	12.8	9.6	7.6	6.4	5.5	4.8	4.2	3.8	3.5	3.2	2.9	2.7	2.5	2.4		
BSUA 25e	" x ½	10.4	7.8	6.2	5.2	4.4	3.9	3.4	3.1	2.8	2.6	2.4	2.2	2.0	1.9		
BSUA 21f	6 x 4 x ½	16.4	12.3	9.8	8.2	7.0	6.1	5.5	4.9	4.5	4.1	3.8	3.5	3.3	3.1		
BSUA 21e	" x ½	13.4	10.0	8.0	6.7	5.7	5.0	4.6	4.0	3.6	3.3	3.1	2.9	2.7	2.5		
BSUA 20f	6 x 3½ x ½	12.6	9.4	7.5	6.3	5.4	4.7	4.2	3.7	3.4	3.1	2.9	2.7	2.5	2.3		
BSUA 20e	" x ½	10.3	7.7	6.2	5.1	4.4	3.8	3.4	3.1	2.8	2.5	2.4	2.2	2.0	1.9		
BSUA 20d	" x ½	7.8	5.9	4.7	3.9	3.3	2.9	2.6	2.3	2.1	1.9	1.8	1.6	1.5	1.4		
RBUA 63f	6 x 3 x ½	9.2	6.9	5.5	4.6	3.9	3.4	3.0	2.7	2.5	2.3	2.1	1.9	1.8	1.7		
RBUA 63e	" x ½	7.5	5.6	4.5	3.7	3.2	2.8	2.5	2.2	2.0	1.8	1.7	1.6	1.5	1.4		
RBUA 63d	" x ½	5.8	4.3	3.4	2.9	2.5	2.1	1.9	1.7	1.5	1.4	1.3	1.2	1.1	1.0		

NOTE PARTICULARLY that **BREAKING** load values are given in this table, based on an ultimate stress of 30 tons per square inch, and corresponding to axis YY.

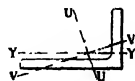
Angles as purlins or side framing bars (usually bolted at each end and continuous over two or more spans) may be stressed safely up to 10 tons per square inch, equal to a factor of safety of 3. For angles as beams over single spans, with ends simply supported, the factor of safety should be 4.

# REDPATH, BROWN & CO., LIMITED.

## STEEL UNEQUAL ANGLES.

Short Leg Vertical.

Dimensions and Properties.



Size, D x B x t inches.	Weight per foot in lbs.	Area in square inches.	Moments of Inertia.			Modulus of Section.	Distrib- uted Breaking Load on 1-ft. Span.	Deflection Coefficient.
			Axis YY.	Axis UU Max.	Axis VV Min.			
7 x 3½ x ¾	24.86	7.313	5.95	37.73	3.90	2.26	45.2	.003552
" x ¾	20.98	6.172	5.15	32.32	3.38	1.92	38.4	.003486
" x ½	17.00	5.000	4.28	26.64	2.74	1.56	31.2	.003422
6 x 4 x ¾	19.92	5.860	7.36	23.83	4.33	2.47	49.4	.003147
" x ½	16.15	4.750	6.10	19.72	3.51	2.02	40.3	.003098
6 x 3½ x ¾	18.87	5.550	4.97	21.77	3.09	1.89	37.8	.003566
" x ½	15.31	4.502	4.14	18.00	2.53	1.55	31.0	.003502
" x ¾	11.64	3.424	3.22	13.83	1.98	1.18	23.6	.003438
6 x 3 x ¾	17.80	5.236	3.13	19.84	2.08	1.38	27.6	.004130
" x ½	14.46	4.252	2.62	16.44	1.68	1.13	22.6	.004041
" x ¾	11.00	3.236	2.05	12.72	1.33	0.87	17.4	.003956

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

All above sections are in our stocks.

For full explanations of tables, see notes commencing page 108.

For formulae, explanations of properties, &c., see Part IV.

Let  $\delta$  = deflection in inches,  $K$  = deflection coefficient,  $L$  = span in feet, and  $F$  = factor of safety, then  $\delta = \frac{K \times L^3}{F}$

# REDPATH, BROWN & CO., LIMITED.



## STEEL UNEQUAL ANGLES.

Short Leg Vertical.

Distributed **BREAKING** Loads, in Tons.

Reference Mark.	Size, D × B × t inches.	SPANS IN FEET.														
		2	3	4	5	6	7	8	9	10	11	12	13	14	15	
BSUA 17f	5 × 4 × $\frac{5}{8}$	24.2	16.1	12.1	9.7	8.0	6.9	6.0	5.4	4.8	4.4	4.0	3.7	3.4	3.2	
BSUA 17e	" × $\frac{1}{2}$	19.8	13.2	9.9	7.9	6.6	5.6	4.9	4.4	3.9	3.6	3.3	3.0	2.8	2.6	
BSUA 17d	" × $\frac{3}{8}$	15.1	10.1	7.5	6.0	5.0	4.3	3.7	3.3	3.0	2.7	2.5	2.3	2.1	2.0	
BSUA 15f	5 × 3 × $\frac{5}{8}$	13.5	9.0	6.7	5.4	4.5	3.8	3.4	3.0	2.7	2.4	2.2	2.1	1.9	1.8	
BSUA 15e	" × $\frac{1}{2}$	11.1	7.4	5.5	4.4	3.7	3.1	2.7	2.4	2.2	2.0	1.8	1.7	1.6	1.5	
BSUA 15d	" × $\frac{3}{8}$	8.5	5.7	4.2	3.4	2.8	2.4	2.1	1.9	1.7	1.5	1.4	1.3	1.2	1.1	
BSUA 11e	4 × 3 × $\frac{1}{2}$	10.9	7.2	5.4	4.3	3.6	3.1	2.7	2.4	2.1	1.9	1.8	1.6	1.5	1.4	
BSUA 11d	" × $\frac{3}{8}$	8.3	5.6	4.2	3.3	2.8	2.4	2.1	1.8	1.6	1.5	1.4	1.3	1.2	1.1	
BSUA 7d	3 × 2½ × $\frac{3}{8}$	5.6	3.7	2.8	2.2	1.8	1.6	1.4	1.2	1.1	1.0	0.9	0.8	0.8	0.7	
BSUA 7c	" × $\frac{1}{4}$	4.7	3.1	2.3	1.9	1.5	1.3	1.1	1.0	0.9	0.8	0.8	0.7	0.7	0.6	

NOTE PARTICULARLY that **BREAKING** load values are given in this table, based on an ultimate stress of 80 tons per square inch, and corresponding to axis YY.

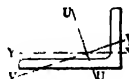
Angles as purlins or side framing bars (usually bolted at each end and continuous over two or more spans) may be stressed safely up to 10 tons per square inch, equal to a factor of safety of 3. For angles as beams over single spans, with ends simply supported, the factor of safety should be 4.

# REDPATH, BROWN & CO., LIMITED.

## STEEL UNEQUAL ANGLES.

• Short Leg Vertical.

Dimensions and Properties.



Size, D × B × t inches.	Weight per foot in lbs.	Area in square inches.	Moments of Inertia.			Modulus of Section.	Distrib- uted Breaking Load on 1-ft. Span.	Deflection Coefficient.
			Axis YY.	Axis UU Max.	Axis VV Min.	Axis YY.	Axis YY.	Axis YY.
5 × 4 × $\frac{3}{8}$	17.80	5.236	7.61	15.84	3.61	2.42	48.5	.003242
" × $\frac{1}{2}$	14.46	4.252	5.83	13.12	3.00	1.98	39.6	.003190
" × $\frac{5}{8}$	11.00	3.236	4.53	10.15	2.34	1.51	30.3	.003139
5 × 3 × $\frac{3}{8}$	15.67	4.609	3.00	12.38	1.88	1.36	27.1	.004239
" × $\frac{1}{2}$	12.75	3.749	2.51	10.30	1.54	1.11	22.2	.004150
" × $\frac{5}{8}$	9.72	2.859	1.97	8.00	1.21	0.85	17.1	.004064
4 × 3 × $\frac{1}{2}$	11.05	3.251	2.37	6.06	1.29	1.09	21.7	.004299
" × $\frac{3}{4}$	8.45	2.485	1.87	4.74	1.02	0.84	16.7	.004206
3 × 2½ × $\frac{3}{8}$	6.53	1.921	1.02	2.12	0.52	0.56	11.2	.005200
" × $\frac{1}{2}$	5.51	1.620	0.87	1.82	0.44	0.48	9.5	.005132

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

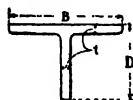
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For full explanations of tables, see notes commencing page 108.

For formulae, explanations of properties, &c., see Part IV.

Let  $\delta$  = deflection in inches, K = deflection coefficient, L = span in feet, and F = factor of safety, then  $\delta = \frac{K \times L^2}{F}$

# REDPATH, BROWN & CO., LIMITED.



## STEEL TEES.

Table Horizontal.

Distributed **BREAKING** Loads, in Tons.

Reference Mark.	Size, B x D x t inches.	SPANS IN FEET.															
		3	4	5	6	7	8	9	10	11	12	13	14	15	16		
BST 21e	6 x 4 x 1/2	13.3	10.0	8.0	6.6	5.7	5.0	4.4	4.0	3.6	3.3	3.0	2.8	2.6	2.5		
BST 20e	6 x 3 x 1/2	7.6	5.7	4.5	3.8	3.2	2.8	2.5	2.2	2.0	1.9	1.7	1.6	1.5	1.4		
BST 20d	" x 3/8	5.8	4.3	3.4	2.9	2.4	2.1	1.9	1.7	1.5	1.4	1.3	1.2	1.1	1.0		
BST 19e	5 x 4 x 1/2	13.0	9.8	7.8	6.5	5.6	4.9	4.3	3.9	3.5	3.2	3.0	2.8	2.6	2.4		
BST 19d	" x 3/8	9.9	7.4	5.9	4.9	4.2	3.7	3.3	2.9	2.7	2.4	2.3	2.1	1.9	1.8		
BST 17e	5 x 3 x 1/2	7.4	5.5	4.4	3.7	3.1	2.7	2.4	2.2	2.0	1.8	1.7	1.5	1.4	1.3		
BST 17d	" x 3/8	5.6	4.2	3.4	2.8	2.4	2.1	1.8	1.7	1.5	1.4	1.3	1.2	1.1	1.0		
BST 16e	4 x 5 x 1/2	19.8	14.9	11.9	9.9	8.5	7.4	6.6	5.9	5.4	4.9	4.5	4.2	3.9	3.7		
BST 16d	" x 3/8	14.6	11.0	8.8	7.3	6.2	5.5	4.8	4.4	4.0	3.6	3.3	3.1	2.9	2.7		

NOTE PARTICULARLY that BREAKING load values are given in this table, based on an ultimate stress of 80 tons per square inch, and corresponding to axis XX.

Tees as purlins or side framing bars (usually bolted at each end and continuous over two or more spans) may be stressed safely up to 10 tons per square inch, equal to a factor of safety of 3. For tees as beams over single spans, with ends simply supported, the factor of safety should be 4.

# REDPATH, BROWN & CO., LIMITED.

## STEEL TEES.

Table Horizontal.

Dimensions and Properties.



Size, B x D x t inches.	Weight per foot in lbs.	Area in square inches.	Moment of Inertia.	Modulus of Section.	Distributed Breaking Load on 1-ft. Span.	Deflection Coefficient.
			Axis XX.	Axis XX.	Axis XX.	Axis XX.
6 x 4 x ½	16.22	4.771	6.07	2.00	40.0	.012377
6 x 3 x ½	14.53	4.272	2.63	1.14	22.8	.016164
" x ¾	11.08	3.260	2.06	0.87	17.4	.015823
5 x 4 x ½	14.51	4.268	5.77	1.96	39.2	.012712
" x ¾	11.07	3.257	4.47	1.49	29.8	.012500
5 x 3 x ½	12.79	3.762	2.52	1.11	22.2	.016593
" x ¾	9.78	2.875	1.97	0.85	17.0	.016234
4 x 5 x ½	14.50	4.264	10.34	2.98	59.6	.010807
" x ¾	11.06	3.253	7.77	2.20	44.0	.010624

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

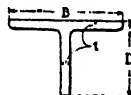
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For full explanations of tables, see notes commencing page 108.

For formulae, explanations of properties, &c., see Part IV.

Let  $\delta$  = deflection in inches,  $K$  = deflection coefficient,  $L$  = span in feet, and  $F$  = factor of safety, then  $\delta = \frac{K \times L^3}{F}$

# REDPATH, BROWN & CO., LIMITED.



## STEEL TEES.

Table Horizontal.

Distributed **BREAKING** Loads, in Tons.

Reference Mark.	Size, B x D x t inches.	SPANS IN FEET.														
		2	3	4	5	6	7	8	9	10	11	12	13	14	15	
BST 15e	4 x 4 x ½	19.0	12.6	9.5	7.6	6.3	5.4	4.7	4.2	3.8	3.4	3.1	2.9	2.7	2.5	
BST 15d	" x ⅝	14.4	9.6	7.2	5.7	4.8	4.1	3.6	3.2	2.8	2.6	2.4	2.2	2.0	1.9	
BST 14e	4 x 3 x ½	10.8	7.2	5.4	4.3	3.6	3.0	2.7	2.4	2.1	1.9	1.8	1.6	1.5	1.4	
BST 14d	" x ⅝	8.3	5.5	4.1	3.3	2.7	2.3	2.0	1.8	1.6	1.5	1.3	1.2	1.1	1.1	
BST 13e	3½ x 3½ x ½	14.4	9.6	7.2	5.7	4.8	4.1	3.6	3.2	2.8	2.6	2.4	2.2	2.0	1.9	
BST 13d	" x ⅝	11.0	7.3	5.5	4.4	3.6	3.1	2.7	2.4	2.2	2.0	1.8	1.6	1.5	1.4	
BST 11e	3 x 3 x ½	10.4	6.9	5.2	4.1	3.4	2.9	2.6	2.3	2.0	1.9	1.7	1.6	1.4	1.3	
BST 11d	" x ⅝	8.0	5.3	4.0	3.2	2.6	2.2	2.0	1.7	1.6	1.4	1.3	1.2	1.1	1.0	
BST 8d	2½ x 2½ x ⅝	5.5	3.6	2.7	2.2	1.8	1.5	1.3	1.2	1.1	1.0	0.9				
BST 8b	" x ¼	3.7	2.4	1.8	1.4	1.2	1.0	0.9	0.8	0.7	0.6	0.6				

NOTE PARTICULARLY that **BREAKING** load values are given in this table, based on an ultimate stress of 30 tons per square inch, and corresponding to axis XX.

Tees as purlins or side framing bars (usually bolted at each end and continuous over two or more spans) may be stressed safely up to 10 tons per square inch, equal to a factor of safety of 3. For tees as beams over single spans, with ends simply supported, the factor of safety should be 4.

# REDPATH, BROWN & CO., LIMITED.

## STEEL TEES.

Table Horizontal.



### Dimensions and Properties.

Size, B x D x t inches.	Weight per foot in lbs.	Area in square inches.	Moment of Inertia.	Modulus of Section.	Distributed Breaking Load on 1-ft. Span.	Deflection Coefficient.
			Axis xx.	Axis xx.	Axis xx.	Axis xx.
4 x 4 x ½	12.78	3.758	5.40	1.90	38.0	.013205
" x ¾	9.77	2.872	4.19	1.44	28.8	.012976
4 x 3 x ½	11.08	3.260	2.37	1.08	21.6	.017202
" x ¾	8.49	2.498	1.86	0.83	16.6	.016817
3½ x 3½ x ½	11.08	3.258	3.54	1.44	28.8	.015244
" x ¾	8.49	2.496	2.77	1.10	22.0	.014941
3 x 3 x ½	9.38	2.760	2.17	1.04	20.8	.018029
" x ¾	7.21	2.121	1.71	0.80	16.0	.017606
2½ x 2½ x ¾	5.92	1.741	0.96	0.55	11.0	.021429
" x ½	4.07	1.197	0.68	0.37	7.4	.020834

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

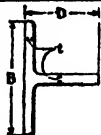
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For formulae, explanation of properties, &c., see Part IV.

Let  $\delta$  = deflection in inches,  $K$  = deflection coefficient,  $L$  = span in feet, and  $F$  = factor of safety, then  $\delta = \frac{K \times L^3}{F}$

# REDPATH, BROWN & CO., LIMITED.



## STEEL TEES.

Stalk Horizontal.

Distributed **BREAKING** Loads, in Tons.

Reference Mark.	Size, B x D x t inches.	SPANS IN FEET.															
		3	4	5	6	7	8	9	10	11	12	13	14	15	16		
BST 21e	6 x 4 x 1/2	19.1	14.3	11.4	9.5	8.2	7.1	6.3	5.7	5.2	4.7	4.4	4.1	3.8	3.5		
BST 20e	6 x 3 x 1/2	19.2	14.4	11.5	9.6	8.2	7.2	6.4	5.7	5.2	4.8	4.4	4.1	3.8	3.6		
BST 20d	" x 3/8	14.2	10.6	8.5	7.1	6.1	5.3	4.7	4.2	3.8	3.5	3.2	3.0	2.8	2.6		
BST 19e	5 x 4 x 1/2	13.4	10.0	8.0	6.7	5.7	5.0	4.4	4.0	3.6	3.3	3.1	2.8	2.6	2.5		
BST 19d	" x 3/8	9.8	7.4	5.9	4.9	4.2	3.7	3.3	2.9	2.7	2.4	2.2	2.1	1.9	1.8		
BST 17e	5 x 3 x 1/2	13.4	10.0	8.0	6.7	5.7	5.0	4.4	4.0	3.6	3.3	3.1	2.8	2.6	2.5		
BST 17d	" x 3/8	9.9	7.4	5.9	4.9	4.2	3.7	3.3	2.9	2.7	2.4	2.3	2.1	1.9	1.8		
BST 16e	4 x 5 x 1/2	8.6	6.4	5.1	4.3	3.6	3.2	2.8	2.5	2.3	2.1	1.9	1.8	1.7	1.6		
BST 16d	" x 3/8	6.2	4.7	3.7	3.1	2.6	2.3	2.0	1.8	1.7	1.5	1.4	1.3	1.2	1.1		

NOTE PARTICULARLY that **BREAKING** load values are given in this table, based on an ultimate stress of 30 tons per square inch, and corresponding to axis YY.

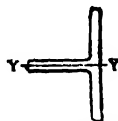
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# REDPATH, BROWN & CO., LIMITED.

## STEEL TEES.

Stalk Horizontal.

Dimensions and Properties.



Size, B x D x t inches.	Weight per foot in lbs.	Area in square inches.	Moment of Inertia.	Modulus of Section.	Distributed Breaking Load on 1-ft. Span.	Deflection Coefficient.
			Axis YY.	Axis YY.	Axis YY.	Axis YY.
6 x 4 x 1/2	16.22	4.771	8.62	2.87	57.4	.003125
6 x 3 x 1/2	14.53	4.272	8.65	2.88	57.6	.003125
" x 3/8	11.08	3.260	6.39	2.13	42.6	.003125
5 x 4 x 1/2	14.51	4.268	5.02	2.01	40.2	.003750
" x 3/8	11.07	3.257	3.69	1.48	29.6	.003750
5 x 3 x 1/2	12.79	3.762	5.03	2.01	40.2	.003750
" x 3/8	9.78	2.875	3.72	1.49	29.8	.003750
4 x 5 x 1/2	14.50	4.264	2.58	1.29	25.8	.004688
" x 3/8	11.06	3.253	1.89	0.94	18.8	.004688

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2 1/2 per cent. over this must be allowed. See page 7.

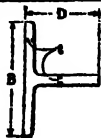
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For formulae, explanations of properties, &c., see Part IV.

Let  $\delta$  = deflection in inches, K = deflection coefficient, L = span in feet, and  $E$  = factor of safety, then  $\delta = \frac{K \times L^3}{E}$

# REDPATH, BROWN & CO., LIMITED.



## STEEL TEES.

Stalk Horizontal.

Distributed **BREAKING** Loads, in Tons.

Reference Mark.	Size, B × D × t inches.	SPANS IN FEET.														
		2	3	4	5	6	7	8	9	10	11	12	13	14	15	
BST 15e	4 × 4 × $\frac{1}{2}$	12.9	8.6	6.4	5.1	4.3	3.6	3.2	2.8	2.5	2.3	2.1	1.9	1.8	1.7	
BST 15d	" × $\frac{3}{8}$	9.5	6.3	4.7	3.8	3.1	2.7	2.3	2.1	1.9	1.7	1.5	1.4	1.3	1.2	
BST 14e	4 × 3 × $\frac{1}{2}$	13.0	8.6	6.5	5.2	4.3	3.7	3.2	2.8	2.6	2.3	2.1	2.0	1.8	1.7	
BST 14d	" × $\frac{3}{8}$	9.6	6.4	4.8	3.8	3.2	2.7	2.4	2.1	1.9	1.7	1.6	1.4	1.3	1.2	
BST 13e	3½ × 3½ × $\frac{1}{2}$	10.0	6.6	5.0	4.0	3.3	2.8	2.5	2.2	2.0	1.7	1.6	1.5	1.4	1.3	
BST 13d	" × $\frac{3}{8}$	7.3	4.8	3.6	2.9	2.4	2.0	1.8	1.6	1.4	1.3	1.2	1.1	1.0	0.9	
BST 11e	3 × 3 × $\frac{1}{2}$	7.4	4.9	3.7	2.9	2.4	2.1	1.8	1.6	1.4	1.3	1.2	1.1	1.0	0.9	
BST 11d	" × $\frac{3}{8}$	5.4	3.6	2.7	2.1	1.8	1.5	1.3	1.2	1.0	0.9	0.9	0.8	0.7	0.7	
BST 8d	2½ × 2½ × $\frac{3}{8}$	3.8	2.5	1.9	1.5	1.2	1.0	0.9	0.8	0.7	0.7	0.6				
BST 8b	" × $\frac{1}{4}$	2.4	1.6	1.2	0.9	0.8	0.6	0.6	0.5	0.5	0.4	0.4				

NOTE PARTICULARLY that **BREAKING** load values are given in this table, based on an ultimate stress of 80 tons per square inch, and corresponding to axis YY.

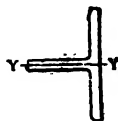
Tees as purlins or side framing bars (usually bolted at each end and continuous over two or more spans) may be stressed safely up to 10 tons per square inch, equal to a factor of safety of 8. For tees as beams over single spans, with ends simply supported, the factor of safety should be 4.

# REDPATH, BROWN & CO., LIMITED.

## STEEL TEES.

Stalk \*Horizontal.

Dimensions and Properties.



Size, B x D x t inches.	Weight per foot in lbs.	Area in square inches.	Moment of Inertia.	Modulus of Section.	Distributed Breaking Load on 1-ft. Span.	Deflection Coefficient.
			Axis YY.	Axis YY.	Axis YY.	Axis YY.
4 x 4 x ½	12.78	3.758	2.59	1.29	25.8	.004688
" x ⅝	9.77	2.872	1.90	0.95	19.0	.004688
4 x 3 x ½	11.08	3.260	2.60	1.30	26.0	.004688
" x ⅝	8.49	2.498	1.91	0.96	19.2	.004688
3½ x 3½ x ½	11.08	3.258	1.75	1.00	20.0	.005357
" x ⅝	8.49	2.496	1.28	0.73	14.6	.005357
3 x 3 x ½	9.38	2.760	1.12	0.74	14.8	.006250
" x ⅝	7.21	2.121	0.82	0.54	10.8	.006250
2½ x 2½ x ⅝	5.92	1.741	0.47	0.38	7.6	.007500
" x ½	4.07	1.197	0.30	0.24	4.8	.007500

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

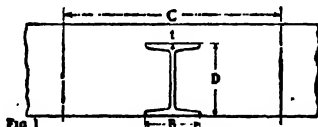
All above sections are in our stocks.

For full explanations of tables, see notes commencing page 108.

For formulae, explanations of properties, &c., see Part IV.

Let  $\delta$  = deflection in inches,  $K$  = deflection coefficient,  $L$  = span in feet, and  $F$  = factor of safety, then  $\delta = \frac{K \times L^3}{F}$

# REDPATH, BROWN & CO., LIMITED.



## STEEL JOISTS.

Embedded in Concrete.

Safe Distributed Loads, in Tons.

Steel Joist.		Concrete thickness "t" in ins.	SPANS IN FEET.											
Size, D x B inches.	Weight per foot in lbs.		6			7			8			9		
			CONCRETE WIDTH "C" IN INCHES.											
			24	18	12	24	18	12	24	18	12	24	18	12
10 x 5	30	2	33-9	31-3	29-1	29-0	26-8	24-9	25-4	23-4	21-8	22-6	20-8	19-4
		1	32-3	30-3	28-3	27-7	26-0	24-2	24-2	22-7	21-2	21-5	20-2	18-9
		0	30-7	29-1	27-5	26-3	24-9	23-6	23-0	21-8	20-6	20-5	19-4	18-3
9 x 4	21	2	23-3	21-2	19-1	19-9	18-2	16-4	17-4	15-9	14-3	15-5	14-1	12-7
		1	21-7	20-0	18-4	18-6	17-1	15-7	16-2	15-0	13-8	14-5	13-4	12-2
		0	20-2	18-9	17-6	17-3	16-2	15-1	15-2	14-2	13-2	13-5	12-6	11-7
8 x 6	35	2	29-8	28-2	26-4	25-6	24-1	22-7	22-4	21-1	19-8	19-9	18-8	17-6
		1	28-6	27-2	25-8	24-5	23-3	22-1	21-4	20-4	19-4	19-0	18-1	17-2
		0	27-3	26-2	25-2	23-4	22-5	21-5	20-5	19-7	18-8	18-2	17-5	16-7
8 x 5	28	2	25-5	23-8	22-1	21-9	20-4	18-9	19-1	17-8	16-5	17-0	15-8	14-7
		1	24-2	22-8	21-4	20-7	19-5	18-3	18-1	17-1	16-1	16-1	15-2	14-3
		0	22-9	21-8	20-7	19-6	18-7	17-8	17-2	16-4	15-6	15-2	14-5	13-8
8 x 4	18	2	18-4	16-7	15-0	15-8	14-3	12-9	13-8	12-5	11-2	12-2	11-2	10-0
		1	17-1	15-7	14-3	14-6	13-4	12-3	12-8	11-8	10-7	11-4	10-4	9-5
		0	15-7	14-7	13-6	13-5	12-6	11-7	11-8	11-0	10-1	10-5	9-8	9-1
7 x 4	16	2	15-0	13-6	12-2	12-9	11-7	10-5	11-3	10-2	9-1	10-0	9-1	8-1
		1	13-8	12-7	11-6	11-9	10-9	9-9	10-4	9-5	8-7	9-2	8-5	7-7
		0	12-6	11-8	11-0	10-8	10-1	9-4	9-5	8-8	8-2	8-4	7-8	7-3

The above safe distributed loads are for steel and concrete combined, and include the weights of both materials.

Maximum safe working stress for steel in tension = 7.5 tons per square inch.

Maximum safe working stress for concrete in compression = 500 lbs. per square inch.

No allowance is made for concrete in tension.

Safe distributed loads for intermediate values of "t" or "C" may be ascertained by interpolation.

# REDPATH, BROWN & CO., LIMITED.

## STEEL JOISTS.

Embedded in Concrete.

Safe Distributed Loads, in Tons.

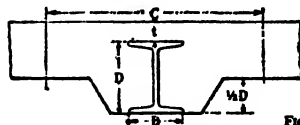


Fig. 2

### SPANS IN FEET.

10	11	12	13	14
----	----	----	----	----

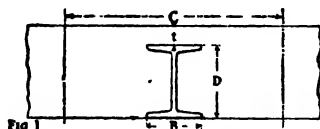
### CONCRETE WIDTH "C" IN INCHES.

24	18	12	24	18	12	24	18	12	24	18	12	24	18	12
20.3	18.7	17.5	18.5	17.0	15.9	17.0	15.6	14.5	15.6	14.4	13.4	14.5	13.4	12.5
19.4	18.2	17.0	17.6	16.5	15.4	16.1	15.1	14.1	14.9	14.0	13.0	13.8	13.0	12.1
18.4	17.5	16.5	16.7	15.9	15.0	15.4	14.5	13.7	14.2	13.4	12.7	13.2	12.5	11.8
14.0	12.7	11.5	12.7	11.6	10.4	11.6	10.6	9.6	10.7	9.8	8.8	10.0	9.1	8.2
13.0	12.0	11.0	11.9	10.9	10.0	10.9	10.0	9.2	10.0	9.3	8.5	9.3	8.6	7.9
12.1	11.4	10.6	11.0	10.3	9.6	10.1	9.5	8.8	9.3	8.7	8.1	8.7	8.1	7.5
17.9	16.9	15.9	16.3	15.4	14.4	14.9	14.1	13.2	13.8	13.0	12.2	12.8	12.1	11.3
17.1	16.3	15.5	15.6	14.8	14.1	14.3	13.6	12.9	13.2	12.5	11.9	12.2	11.6	11.0
16.4	15.7	15.1	14.9	14.3	13.7	13.6	13.1	12.5	12.6	12.1	11.6	11.7	11.2	10.8
15.3	14.3	13.2	13.9	13.0	12.0	12.7	11.9	11.0	11.8	11.0	10.2	10.9	10.2	9.5
14.5	13.7	12.8	13.2	12.4	11.7	12.1	11.4	10.7	11.2	10.5	9.9	10.4	9.8	9.2
13.7	13.1	12.4	12.5	11.9	11.3	11.4	10.9	10.4	10.6	10.1	9.6	9.8	9.3	8.9
11.1	10.0	9.0	10.1	9.0	8.2	9.2	8.4	7.5	8.5	7.7	6.9	7.9	7.2	6.4
10.2	9.4	8.6	9.3	8.5	7.8	8.5	7.8	7.2	7.9	7.2	6.6	7.3	6.7	6.1
9.4	8.8	8.2	8.6	8.0	7.4	7.9	7.3	6.8	7.3	6.8	6.3	6.7	6.3	5.8
9.0	8.2	7.3	8.2	7.4	6.6	7.5	6.8	6.1	6.9	6.3	5.6	6.4	5.8	5.2
8.3	7.6	6.9	7.5	6.9	6.3	6.9	6.3	5.8	6.4	5.9	5.3	5.9	5.4	5.0
7.6	7.1	6.6	6.9	6.4	6.0	6.3	5.9	5.5	5.8	5.4	5.1	5.4	5.0	4.7

In floor construction the total thickness of the concrete between steel joists may be reduced by the value of  $\frac{1}{2}D$  as indicated in Fig. 2,  $D$  being the total depth of steel joist.

This does not affect the tabular safe loads, as in the calculation of these the strength of concrete in tension is neglected, and the neutral axis of the combined beam is always above that of the steel joist.

# REDPATH, BROWN & CO., LIMITED.



## STEEL JOISTS.

Embedded in Concrete.

Safe Distributed Loads, in Tons.

Steel Joist.			SPANS IN FEET.															
Size, D x B inches.	Weight per foot in lbs.	Con- crete thick- ness "t" in ins.	6				7				8				9			
			CONCRETE WIDTH "C" IN INCHES.															
			18	15	12	18	15	12	18	15	12	18	15	12				
7 x 2½	12	2	9.6	8.9	8.1	8.3	7.6	6.9	7.2	6.6	6.1	6.4	5.9	5.4				
		1	8.7	8.1	7.5	7.4	6.9	6.4	6.5	6.0	5.6	5.8	5.4	5.0				
		0	7.7	7.3	6.8	6.6	6.2	5.8	5.8	5.4	5.1	5.2	4.8	4.6				
6 x 5	25	2	15.3	14.7	14.2	13.2	12.7	12.2	11.5	11.1	10.7	10.2	9.8	9.5				
		1	14.6	14.2	13.8	12.5	12.1	11.8	10.9	10.6	10.3	9.7	9.4	9.2				
		0	13.9	13.6	13.3	11.9	11.6	11.4	10.4	10.2	10.0	9.3	9.0	8.8				
6 x 4½	20	2	12.8	12.2	11.7	11.0	10.5	10.1	9.6	9.2	8.8	8.5	8.1	7.8				
		1	12.1	11.6	11.2	10.3	9.9	9.6	9.0	8.7	8.4	8.0	7.7	7.5				
		0	11.3	11.0	10.7	9.7	9.4	9.2	8.5	8.3	8.1	7.5	7.3	7.2				
6 x 3	12	2	8.9	8.4	7.8	7.6	7.2	6.7	6.7	6.3	5.9	5.9	5.6	5.2				
		1	8.1	7.7	7.3	7.0	6.6	6.2	6.1	5.8	5.5	5.4	5.1	4.9				
		0	7.3	7.0	6.8	6.3	6.0	5.8	5.5	5.3	5.1	4.9	4.7	4.5				
5½ x 2	9½	2	6.7	6.0	5.4	5.7	5.2	4.6	5.0	4.5	4.0	4.4	4.0	3.6				
		1	5.8	5.3	4.8	5.0	4.6	4.2	4.4	4.0	3.6	3.9	3.6	3.2				
		0	5.0	4.7	4.3	4.3	4.0	3.7	3.8	3.5	3.2	3.3	3.1	2.9				
5 x 4½	18	2	10.2	9.7	9.3	8.7	8.3	8.0	7.6	7.3	7.0	6.8	6.5	6.2				
		1	9.5	9.2	8.9	8.2	7.9	7.6	7.2	6.9	6.7	6.4	6.1	5.9				
		0	8.9	8.6	8.4	7.6	7.4	7.2	6.7	6.5	6.3	5.9	5.7	5.5				

The above safe distributed loads are for steel and concrete combined, and include the weights of both materials.

Maximum safe working stress for steel in tension = 7.5 tons per square inch.

Maximum safe working stress for concrete in compression = 500 lbs. per square inch.

No allowance is made for concrete in tension.

Safe distributed loads for intermediate values of "t" or "C" may be ascertained by interpolation.

# REDPATH, BROWN & CO., LIMITED.

## STEEL JOISTS.

Embedded in Concrete.

Safe Distributed Loads, in Tons.

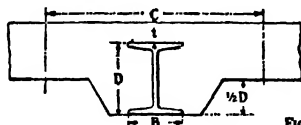


Fig 2

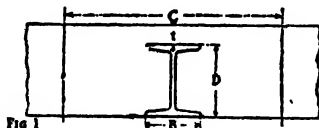
### SPANS IN FEET.

10			11			12			13			14		
CONCRETE WIDTH "C" IN Inches.														
18	15	12	18	15	12	18	15	12	18	15	12	18	15	12
5.8	5.3	4.9	5.3	4.8	4.4	4.8	4.4	4.1	4.5	4.1	3.7	4.1	3.8	3.5
5.2	4.8	4.5	4.7	4.4	4.1	4.3	4.0	3.7	4.0	3.7	3.4	3.7	3.5	3.2
4.6	4.4	4.1	4.2	4.0	3.7	3.9	3.6	3.4	3.6	3.2	3.2	3.3	3.1	2.9
9.2	8.9	8.6	8.4	8.1	7.8	7.7	7.4	7.1	7.1	6.8	6.6	6.6	6.3	6.1
8.8	8.5	8.3	8.0	7.7	7.5	7.3	7.1	6.9	6.7	6.5	6.4	6.3	6.1	5.9
8.3	8.1	8.0	7.6	7.4	7.2	6.9	6.7	6.6	6.4	6.2	6.1	6.0	5.8	5.7
7.7	7.3	7.0	7.0	6.7	6.4	6.4	6.1	5.9	5.9	5.6	5.4	5.5	5.2	5.0
7.2	6.9	6.7	6.6	6.3	6.1	6.0	5.8	5.6	5.6	5.4	5.2	5.2	5.0	4.8
6.8	6.6	6.4	6.2	6.0	5.9	5.7	5.5	5.4	5.2	5.1	5.0	4.9	4.7	4.6
5.4	5.0	4.7	4.9	4.6	4.3	4.5	4.2	3.9	4.1	3.9	3.6	3.8	3.6	3.3
4.9	4.6	4.4	4.4	4.2	4.0	4.1	3.8	3.6	3.7	3.6	3.4	3.5	3.3	3.1
4.4	4.2	4.1	4.0	3.8	3.7	3.7	3.5	3.4	3.4	3.2	3.1	3.1	3.0	2.9
4.0	3.6	3.2	3.6	3.3	2.9	3.3	3.0	2.7	3.1	2.8	2.5	2.8	2.6	2.3
3.5	3.2	2.9	3.2	2.9	2.6	2.9	2.7	2.4	2.7	2.5	2.2	2.5	2.3	2.1
3.0	2.8	2.6	2.7	2.5	2.3	2.5	2.3	2.1	2.3	2.1	2.0	2.1	2.0	1.8
6.1	5.8	5.6	5.6	5.3	5.1	5.1	4.9	4.7	4.7	4.5	4.3	4.4	4.2	4.0
5.7	5.5	5.3	5.2	5.0	4.8	4.8	4.6	4.4	4.4	4.2	4.1	4.1	3.9	3.8
5.3	5.2	5.1	4.9	4.7	4.6	4.4	4.3	4.2	4.1	4.0	3.9	3.8	3.7	3.6

In floor construction the total thickness of the concrete between steel joists may be reduced by the value of  $\frac{1}{2}D$  as indicated in Fig. 2, D being the total depth of the steel joist.

This does not affect the tabular safe loads, as in the calculation of these the strength of concrete in tension is neglected, and the neutral axis of the combined beam is always above that of the steel joist.

# REDPATH, BROWN & CO., LIMITED.



## STEEL JOISTS.

Embedded in Concrete.

Safe Distributed Loads, in Tons.

Steel Joist.		Concrete thickness "t" in ins.	SPANS IN FEET.											
Size, D x B inches.	Weight per foot in lbs.		5			6			7			8		
			CONCRETE WIDTH "C" IN INCHES.											
			18	15	12	18	15	12	18	15	12	18	15	12
5 x 3	11	2	8.6	8.1	7.5	7.2	6.7	6.3	6.1	5.8	5.4	5.4	5.0	4.7
		1	7.8	7.4	7.0	6.5	6.1	5.8	5.5	5.3	5.0	4.9	4.6	4.4
		0	6.9	6.7	6.4	5.8	5.6	5.4	5.0	4.8	4.6	4.3	4.2	4.0
4½ x 1½	6½	2	5.9	5.4	4.9	4.9	4.5	4.0	4.2	3.8	3.5	3.7	3.4	3.0
		1	5.0	4.6	4.3	4.2	3.9	3.6	3.6	3.3	3.1	3.1	2.9	2.7
		0	4.1	3.9	3.7	3.4	3.3	3.1	2.9	2.8	2.6	2.6	2.4	2.3
4 x 3	9½	2	6.1	5.7	5.3	5.1	4.7	4.4	4.3	4.1	3.8	3.8	3.6	3.3
		1	5.4	5.1	4.9	4.5	4.3	4.0	3.9	3.7	3.5	3.4	3.2	3.0
		0	4.7	4.6	4.4	3.9	3.8	3.7	3.4	3.3	3.1	3.0	2.9	2.8
4 x 1½	5	2	4.3	3.9	3.5	3.6	3.2	2.9	3.1	2.8	2.5	2.7	2.4	2.2
		1	3.5	3.2	2.9	2.9	2.7	2.4	2.5	2.3	2.1	2.2	2.0	1.8
		0	2.7	2.6	2.4	2.3	2.1	2.0	1.9	1.8	1.7	1.7	1.6	1.5
3 x 3	8½	2	4.1	3.9	3.6	3.5	3.2	3.0	3.0	2.8	2.6	2.6	2.4	2.2
		1	3.6	3.4	3.3	3.0	2.9	2.7	2.6	2.5	2.3	2.3	2.1	2.0
		0	3.1	3.0	2.9	2.6	2.5	2.4	2.2	2.1	2.1	2.0	1.9	1.8
3 x 1½	4	2	2.9	2.6	2.3	2.4	2.2	1.9	2.1	1.9	1.6	1.8	1.6	1.4
		1	2.2	2.0	1.9	1.9	1.7	1.5	1.6	1.5	1.3	1.4	1.3	1.1
		0	1.6	1.5	1.4	1.3	1.2	1.2	1.1	1.1	1.0	1.0	0.9	0.9

The above safe distributed loads are for steel and concrete combined, and include the weights of both materials.

Maximum safe working stress for steel in tension = 7.5 tons per square inch.

Maximum safe working stress for concrete in compression = 500 lbs. per square inch.

No allowance is made for concrete in tension.

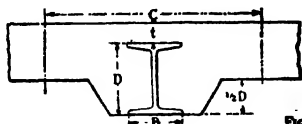
Safe distributed loads for intermediate values of "t" and "C" may be ascertained by interpolation.

# REDPATH, BROWN & CO., LIMITED.

## STEEL JOISTS.

Embedded in Concrete.

Safe Distributed Loads, in Tons.



### SPANS IN FEET.

9	10	11	12	14
---	----	----	----	----

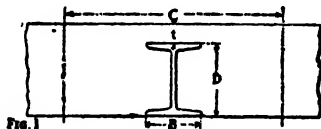
### CONCRETE WIDTH "C" IN INCHES.

18	15	12	18	15	12	18	15	12	18	15	12	18	15	12
4.8	4.5	4.2	4.3	4.0	3.8	3.9	3.7	3.4	3.6	3.4	3.1	3.1	2.9	2.7
4.3	4.1	3.9	3.9	3.7	3.5	3.5	3.3	3.2	3.2	3.1	2.9	2.8	2.6	2.5
3.9	3.7	3.6	3.5	3.3	3.2	3.2	3.0	2.9	2.9	2.8	2.7	2.5	2.4	2.3
3.3	3.0	2.7	2.9	2.7	2.4	2.7	2.4	2.2	2.4	2.2	2.0	2.1	1.9	1.7
2.8	2.6	2.4	2.5	2.3	2.1	2.3	2.1	1.9	2.1	1.9	1.8	1.8	1.7	1.5
2.3	2.2	2.1	2.1	2.0	1.8	1.9	1.8	1.7	1.7	1.6	1.5	1.5	1.4	1.3
3.4	3.2	2.9	3.0	2.8	2.7	2.8	2.6	2.4	2.5	2.4	2.2	2.2	2.0	1.9
3.0	2.8	2.7	2.7	2.6	2.4	2.5	2.3	2.2	2.2	2.1	2.0	1.9	1.8	1.7
2.6	2.5	2.4	2.4	2.3	2.2	2.2	2.1	2.0	2.0	1.9	1.8	1.7	1.6	1.6
2.4	2.1	1.9	2.1	1.9	1.7	1.9	1.8	1.6	1.8	1.6	1.4	1.5	1.4	1.2
1.9	1.8	1.6	1.7	1.6	1.5	1.6	1.5	1.3	1.4	1.3	1.2	1.2	1.1	1.0
1.5	1.4	1.3	1.3	1.3	1.2	1.2	1.2	1.1	1.1	1.1	1.0	1.0	0.9	0.8
2.3	2.1	2.0	2.1	1.9	1.8	1.9	1.8	1.6	1.7	1.6	1.5	1.5	1.4	1.3
2.0	1.9	1.8	1.8	1.7	1.6	1.7	1.6	1.5	1.5	1.4	1.3	1.3	1.2	1.1
1.7	1.7	1.6	1.5	1.5	1.4	1.4	1.4	1.3	1.3	1.2	1.2	1.1	1.0	1.0
1.6	1.4	1.3	1.4	1.3	1.1	1.3	1.2	1.0	1.2	1.1	0.9	1.0	0.9	0.8
1.2	1.1	1.0	1.1	1.0	0.9	1.0	0.9	0.8	0.9	0.8	0.7	0.8	0.7	0.6
0.9	0.8	0.8	0.8	0.7	0.7	0.7	0.7	0.6	0.7	0.6	0.6	0.6	0.5	0.5

In floor construction the total thickness of the concrete between steel joists may be reduced by the value of  $\frac{1}{2}D$  as indicated in Fig. 2,  $D$  being the total depth of the steel joist.

This does not affect the tabular safe loads, as in the calculation of these the strength of concrete in tension is neglected, and the neutral axis of the combined beam is always above that of the steel joist.

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## STEEL JOISTS.

Embedded in Concrete.

Values of "p" and "q"  
see page 107 for application.

Steel Joist.		Strength Increases per cent.		Steel Joist.		Strength Increases per cent.	
Size, D x B inches.	Weight per foot in lbs.	Per inch width of concrete = p.	For 1-in. width per inch depth of concrete above flange = q.	Size, D x B inches.	Weight per foot in lbs.	Per inch width of concrete = p.	For 1-in. width per inch depth of concrete above flange = q.
10 x 5	30	1.1075	0.2743	6 x 3	12	1.6719	0.7912
9 x 4	21	1.4521	0.4178	5½ x 2	9½	4.1938	1.5830
8 x 6	35	0.7681	0.2327	5 x 4½	18	0.9865	0.4781
8 x 5	28	0.9538	0.2939	5 x 3	11	1.5457	0.8414
8 x 4	18	1.4846	0.4867	4½ x 1½	6½	2.5338	1.7050
7 x 4	16	1.4717	0.5428	4 x 3	9½	1.4478	0.9993
7 x 2½	12	2.8930	1.0475	4 x 1½	5	2.7226	2.3988
6 x 5	25	0.8103	0.3383	3 x 3	8½	1.3020	1.1476
6 x 4½	20	0.9878	0.4315	3 x 1½	4	2.5375	3.2781

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## STEEL JOISTS.

Embedded in Concrete.

Application of "p" and "q" values on page 106,  
and example for Tables on pages 100 to 105.

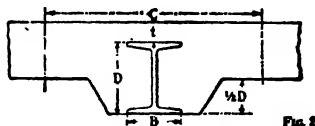


FIG. 2

Safe loads on combined steel and concrete beams for any width or depth of concrete over flange, not given on pages 100 to 105, may be calculated by the percentage values on page 106 opposite.

### FORMULA:

C = total width of concrete, in inches, assumed as acting with steel joist.  
t = total depth of concrete, in inches, above top flange of steel joist.  
p = strength increase per cent. per inch width of concrete, from page 106.  
q = strength increase per cent. for 1-in. width of concrete per inch depth of concrete above top flange of steel joist, from page 106.

Ws = tabular safe load in tons for required span in feet for steel joist only, from pages 16 to 19.

W = safe distributed load in tons on combined steel and concrete beam for same span.

$$W = W_s + \frac{W_s (p \times C + q \times C \times t)}{100}$$

### EXAMPLE:

An 8-in. x 4-in. steel joist will support a safe distributed load of 5.8 tons on a clear span of 12 feet. See table, page 18. Required the safe distributed load in tons on the same span for same section if combined with concrete 17 inches wide and 2½ inches thick over top flange.

C = 17 in.; t = 2½ in.; p = 1.4846, and q = 0.4867 from page 106. Ws = 5.8 tons from page 18.

$$W = 5.8 + \frac{5.8 (1.4846 \times 17 + 0.4867 \times 17 \times 2.5)}{100}$$

$$= 5.8 + 2.663$$

$$= 8.463 \text{ tons.}$$

= total safe distributed load, in tons, for combined beam  
\* on clear span of 12 feet.

Example of the use of Tables, pages 100 to 105 inclusive.

Required the safe distributed load in tons over 9 feet span for a 6-in. x 3-in. steel joist combined with 15 inches width of concrete, and 1 inch thickness over top flange.

See page 102. Read answer 5.1 tons in column "t" = 1 inch for this section, and below span = 9 feet, and "C" = 15 inches.

## PART I.

### Explanations of the Tables.

Pages 16 to 107 inclusive.

See Part IV. for general formulæ for the flexure of beams, explanations of properties, &c.

**Part I.**  
**Arrangement.**

All the tables in this part relate to simple or compound sections, as Beams or Girders.

Steel Joists and Compound Girders formed of these with plates on each flange are the subjects of the tables at the commencement of this part, these being the sections most commonly used.

They are followed by the tables relating to Steel Joists plated on one flange only, Steel Channels, Channel Compound Girders, Plate Girders, Angles and Tees, &c.

**Compound  
Girders.**  
**Pages 20 to 49.**

The selection of Compound Girders is very comprehensive, and the practical limitations due to web buckling, deflection or rivet pitch are clearly indicated by zigzag lines or *italics*.

The usefulness of this large range of Compound Girders is increased by the tables of "Rivet Pitches" and "Moments of Resistance," pages 50 to 67 inclusive.

**Rivet Pitch.**  
**Pages 50 to 59.**

Unless specified otherwise, Compound Girders are riveted at 6 inches pitch. On pages 20 to 49, and page 74, certain loads are printed in *italics*, indicating that the standard pitch of 6 inches is inadequate. For such cases reference should be made to the "Rivet Pitch" tables, pages 50 to 59, from which the required pitch can be readily ascertained.

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On pages 60 to 67, the single, double, and triple beam types of Compound Girders are collected and arranged in descending order of carrying capacity, the criterion being the value of the "Maximum Moment of Resistance" of the section in foot tons. For equilibrium this value must equal the "Maximum Bending Moment" in foot tons, therefore when the latter quantity has been calculated for any regular or irregular system of loading, the various types of girders suitable either with regard to economy or overall dimensions can be seen at once on reference being made to corresponding values in the columns of "Maximum Moments of Resistance, Foot Tons." The weights per foot of girders of equal strength vary considerably, and it may be noted that where the depth of a section is not restricted, one having a "Moment of Resistance" appreciably in excess of the "Bending Moment" may be found more economical.

Moments of  
Resistance.  
Pages 60 to 67.

*Example* :—A system of loading produces a maximum bending moment of 283 foot tons. What is the lightest section of girder suitable?

See page 61. The nearest corresponding maximum moment of resistance is 283·8 foot tons for a girder of 369½ lbs. per foot and 15 inches depth. Reading up the same columns, the moments of resistance increase, but lighter weights per foot may be observed for sections of greater depth, the minimum of 206 lbs. being found opposite a resistance moment of 321·2 foot tons, the girder being 26½ inches deep.

Before deciding finally on any particular section, the tables of safe loads, pages 20 to 49, should be referred to

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in order to see that the deflection, web buckling, and rivet pitch limitations are complied with.

**Special Compound Girders.**  
Pages 68 and 69.

The Compound Girders formed of Steel Joists, plated on one flange only, are useful when for any special purpose it is desirable to have a broad top flange and a narrow bottom flange, or *vice versa*.

Pages 74 and 75.

The Compound Girders formed of two Steel Channels plated on each flange are used chiefly as lintel beams to support walls, relatively broad, but of no great height or weight.

**Plate Girders.**  
Pages 76 to 79.

When Plate Girders are used in building work, they, as a rule, are the subject of special calculations. The selection of Plate Girders on pages 76 to 79 will probably be found sufficient to cover the few cases where greater depth is required than can be got by using Steel Joists plated on each flange.

**Steel Joists in Concrete.**  
Pages 100 to 107.

If Steel Joists are embedded in concrete, as in floors, they are stiffened considerably, and their strength is increased proportionately. The stiffening effect of various widths and thicknesses of concrete is allowed for in the safe distributed loads tabulated on pages 100 to 105. On page 106 there are given percentage increase values per inch width and thickness of concrete, by means of which the safe load for any combination of Joist and concrete may be readily calculated. The application of the tables will be found on page 107.

**Tabular Loads.**

Each tabular load includes the weight of the section or girder itself, and implies uniform distribution over the entire length of the effective span. In building work the "span in feet" is usually taken as the length between

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supports, but the calculations are based on the "effective Tabular Loads. span," i.e., the distance between centres of bearings.

It is assumed that each section or girder has its ends simply supported, not fixed, and is efficiently stayed *laterally* at distances not exceeding 30 times the compression flange width.

*Safe loads* uniformly distributed are tabulated for each Joist, Channel, Compound and Plate Girder. These are based on a safe working stress of 7.5 tons per square inch at the extreme fibres, corresponding to a factor of safety of 4, taking the mean ultimate tensile strength of the steel at 30 tons per square inch.

**Safe Loads.**  
Pages 16 to 49  
and 68 to 79.

*Breaking loads* uniformly distributed are tabulated for each Equal and Unequal Angle and Tee. These are based on an ultimate stress of 30 tons per square inch at the extreme fibres. It is necessary, therefore, in every case that the tabular *breaking* loads be reduced to safe loads by the use of a suitable factor of safety. The *breaking* load of an angle or tee is a more useful value than the safe load based on 7.5 tons per square inch, as the conditions of their use frequently permit of the adoption of a considerably higher stress.

**Breaking Loads.**  
Pages 80 to 99.

The methods of adapting the tables to meet conditions differing from those on which the calculations are based are explained in Part IV.

**Special Conditions.**

All dimensions of sections are stated in inches, and all properties in inch units.

**Dimensions and Properties.**

D = depth, B = breadth, and t = thickness.

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### Composition of Girders.

The composition of Compound Girders is described in the first two columns of the right-hand pages of the tables. The first column shows the number of Sections used, the second column gives the overall width and total thickness of the plates on each flange. When the latter exceeds  $\frac{3}{4}$  of an inch, plates of convenient thicknesses are used to make up the total required. For instance, a total thickness of  $1\frac{1}{2}$  inches may be formed of two  $\frac{3}{4}$  inch or three  $\frac{1}{2}$  inch plates.

The composition of Plate Girders is described in the first three columns of the right-hand pages of the tables. The first column gives the overall depth and thickness of the web or webs, the second column the overall width and total thickness of the plates on each flange, and the third column the section and thickness of the angles.

### Weights per Foot.

The weights per foot in lbs. include an allowance for rivet heads at the respective pitches of 6 inches for Compound Girders and 4 inches for Plate Girders. The weights of stiffeners, end angles, &c., require to be added.

### Areas.

Each area in square inches is the superficial area of a cross section at right angles to the longitudinal axis. No deduction is made for rivet holes in the areas of Compound or Plate Girders, this being allowed for in the calculation of "Moments of Inertia," which see.

### Standard Thicknesses.

These standard thicknesses, pages 17, 19, 71, and 73, are those recommended by the Engineering Standards Committee. It may be noted that the mean flange thicknesses given in these tables are approximately equal to the flange thicknesses of the beams at rivet holes in Compound Girders.

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Maximum and minimum moments of inertia are tabulated for each Joist, Channel, and Tee; the maximum moment only for each Compound and Plate Girder. Moments of Inertia.

For each Joist, Channel, Compound and Plate Girder, the maximum moment of inertia is about "Axis X—X" passing through the centre of gravity of the figure and parallel to the flanges. The minimum moment of inertia for these sections is about "Axis Y—Y" passing through the centre of gravity of the figure and parallel to the web or webs. The tabulated safe loads are relative to the maximum moment of inertia, about "Axis X—X."

The tabulated moments of inertia of each Tee are also about central axes, but the maximum moment may be about "Axis X—X" parallel to the table or "Axis Y—Y" parallel to the stalk, depending upon the dimensions of the section. *Breaking* loads are given relative to both moments, maximum and minimum.

For each Angle, four moments of inertia are given about axes passing through the centre of gravity of the figure. These are the maximum and minimum moments about the minor "Axis U—U" and major "Axis V—V" of the inertia ellipse, also the moments about the Axes "X—X" and "Y—Y" parallel to the legs. In the case of equal angles the moments about axes "X—X" and "Y—Y" parallel to the legs are identical. The tabulated *breaking* loads are *not* relative to the maximum and minimum moments, but to the moments about axes "X—X" and "Y—Y." This implies that either leg of the section is at right angles to the direction of the loading.

The moments of inertia of Compound and Plate

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Girders are calculated on the net sectional area of the figure, a deduction for rivet holes being made.

### Moduli of Sections.

For each Joist, Channel, Compound and Plate Girder, the maximum modulus of section "Axis X—X" corresponding to the maximum moment of inertia is given. These values are printed in prominent type, as they are referred to frequently.

For each Angle and Tee, two section moduli are given. These correspond to the moments of inertia, having the same axial reference letters, thus, "Axis X—X" or "Axis Y—Y."

### Loads on 1 foot Span.

The safe distributed load on 1 foot span is tabulated for each section and also for 1 inch width of flange plates. The load on any span is equal to the load on 1 foot span divided by the required span in feet.

The values "for 1 inch plate width" are the variations of the safe distributed loads, in tons, on 1 foot span, for each inch increase or decrease in the width of flange plates. Corresponding section modulus variations may be got by dividing the safe load variations by 5.

These values are convenient for calculating the strength of girders varying slightly in flange plate width from the sections for which complete properties and safe loads are given. They are to be added or subtracted as width is increased or decreased.

*Example :—*Required safe distributed load in tons over 28 feet span, also section modulus for compound girder 280A, 25 × 12, page 20, with 14 inch instead of 12 inch flange plates.

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Safe load on 1 foot span for girder, -	=	1623 tons.	Loads on 1 foot Span.
Variation for 1 inch plate width =			
60 tons $\therefore$ for two inches, -	-	=	120 "
Total safe load on 1 foot span, -	-	=	<u>1743</u> "
Safe load on 28 feet span, -	=	$\frac{1743}{28}$	= 62.2 "
Section modulus for 2 inches			
variation of plate width, -	=	$\frac{120}{5}$	= 24 inches. <sup>3</sup>
Section modulus of girder			
with 14 inch flange plates, -	=	$\frac{1743}{5}$	= 348.6 "

Before varying the tabulated sections of girders inquiries should be made to ascertain if the desired new widths of flange plates are obtainable, as otherwise inconvenience and delay may follow.

The dotted zigzag lines indicate the limiting loads for shear or web buckling. They are placed to the immediate left of the minimum spans over which the sections can support the *full tabular loads* without provision being made to stiffen the webs. If sections are used over shorter spans:--(1) web stiffeners must be provided, suitable forms of which are shown in Part V., or (2) the maximum loads must not exceed the tabular loads to the immediate right of the lines.

Dotted Zigzag  
Lines.

Dotted zigzag lines are not inserted in the Plate Girder tables, it being assumed that girders of these types will require web stiffeners in accordance with usual practice. The design and spacing of these stiffeners will depend on

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**Dotted Zigzag Lines.**

the depth and thickness of the webs and the system of loading.

**Full Zigzag Lines.**

The full zigzag lines are placed to the immediate right of the maximum spans over which the sections can support the full tabular loads without the deflection exceeding 1/26th of an inch per foot of span. For a symmetrical section, this limit corresponds to 24 times its depth in inches, and for an unsymmetrical section to 48 times the distance in inches from the neutral axis to the extreme fibre.

If sections are intended to support plastered ceilings, it is advisable that the deflection should not exceed 1/32nd of an inch per foot of span. This limit will be complied with if the maximum span for the full tabular load corresponds to 20 times the depth in inches of a symmetrical section, and to 40 times the distance in inches from the neutral axis to the extreme fibre of an unsymmetrical section.

The distances in inches from the neutral axis to the extreme fibre of unsymmetrical sections will be found in Part V.

**Deflection Coefficients.**

A deflection co-efficient, derived from the general deflection formula for uniformly distributed loading, is given for each section.

The formula for its use is as follows :—

$$\left. \begin{array}{l} \delta = \text{deflection in inches,} \\ K = \text{deflection coefficient,} \\ L = \text{span in feet,} \end{array} \right\} \delta = K \times L^2.$$

Deflection in inches = deflection coefficient  $\times$  the square of the span in feet.

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This simplified deflection formula is only applicable under the conditions for which the safe uniformly distributed loads in the tables are calculated, viz. :—

**Deflection  
Coefficients.**

That the modulus of elasticity of the steel is 12,000 tons (average) per square inch.

That the ends of the section are simply supported, not fixed.

That the full tabular load is carried.

As uniformly distributed breaking loads are given for Angles and Tees, the formula for the use of the deflection coefficient is modified by the insertion of a factor of safety as follows :—

$$\left. \begin{array}{l} \delta = \text{deflection in inches,} \\ K = \text{deflection coefficient,} \\ L = \text{span in feet,} \\ F = \text{factor of safety,} \end{array} \right\} \delta = \frac{K \times L^3}{F}$$

Deflection in inches = deflection coefficient  $\times$  the square of the span in feet and  $\div$  the factor of safety.

Suitable factors of safety are indicated on the tables.

The methods of adapting the deflection coefficients to meet special conditions of loading and limited allowable deflection are explained in Part IV.



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**PART II.**  
**STANCHIONS.**

**SAFE LOADS**  
**(BY MONCRIEFF FORMULAE)**

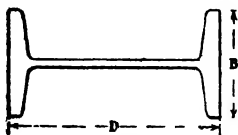
**AND**  
**PROPERTIES,**  
**Etc.**



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## STANCHIONS.

### Steel Joists.

Safe Concentric Loads, in Tons.  
Ends Flat.

Reference Mark.	Size, D x B inches	HEIGHTS IN FEET															
		8	9	10	11	12	13	14	15	16	17	18	20	22	24		
30 J	24 x 7½	182	179	175	170	165	160	145	126	111	98	87	77	71	0		
29 J	20 x 7½	163	160	156	152	148	144	136	118	104	92	182	266	5			
28 J	19 x 7	136	133	130	126	122	117	101	88	077	368	561	1				
27 J	16 x 6	108	105	101	95	279	968	158	751	145	0						
26 J	15 x 6	104	101	98	094	383	270	961	253	346	841	5					
25 J	15 x 5	68	862	650	741	935	230	0									
24 J	14 x 6a	101	98	395	191	682	570	360	652	846	441	1					
23 J	14 x 6b	81	378	976	273	363	754	346	840	835	8						
22 J	12 x 6a	96	593	091	188	083	471	161	353	446	941	5					
21 J	12 x 6b	78	476	273	871	265	756	048	342	036	932	7					
20 J	12 x 5	53	150	641	434	228	824	5									
19 J	10 x 8	131	129	127	125	123	121	118	115	112	105	93	976	162	952	8	
18 J	10 x 6	75	473	571	469	066	557	749	743	338	133	730	1				
17 J	10 x 5	50	448	141	634	428	924	621	2								
16 J	9 x 7	107	105	103	101	98	896	293	487	576	968	160	740	2			

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160.

Safe loads are calculated by the Moncrieff Formula for stanchions of mild steel having "both ends flat."

Safe loads for the condition of "both ends fixed" are identical with tabular loads on the heights to left of signing line.

For other conditions and formulae, see notes commencing page 192.

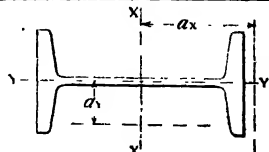
For explanations of properties, &c., see Part IV.

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## STANCHIONS.

### Steel Joists.

#### Dimensions and Properties.



Size, D x B inches.	Weight per foot in lbs.	Area in square inches.	Standard Thicknesses.		Radii of Gyration.		Eccentricity Coefficients.			
			Web.	Flange.	Axis Y-Y	Axis X-X	Web.	Flange.	Axis Y-Y	Axis X-X
24 x 7½	100	29.392	.600	1.070	1.51	9.50	1.50	2.60	1+1.65a <sub>v</sub>	1+0.13a <sub>x</sub>
20 x 7½	89	26.164	.600	1.010	1.54	7.99	1.47	2.57	1+1.57a <sub>v</sub>	1+0.16a <sub>x</sub>
18 x 7	75	22.066	.550	.928	1.45	7.22	1.46	2.56	1+1.66a <sub>v</sub>	1+0.17a <sub>x</sub>
16 x 6	62	18.227	.550	.847	1.22	6.31	1.56	2.61	1+2.02a <sub>v</sub>	1+0.20a <sub>x</sub>
15 x 6	59	17.346	.500	.880	1.27	6.02	1.46	2.55	1+1.85a <sub>v</sub>	1+0.21a <sub>x</sub>
15 x 5	42	12.351	.420	.647	0.98	5.89	1.55	2.62	1+2.59a <sub>v</sub>	1+0.22a <sub>x</sub>
14 x 6a	57	16.769	.500	.873	1.29	5.64	1.45	2.54	1+1.80a <sub>v</sub>	1+0.22a <sub>x</sub>
14 x 6b	46	13.533	.400	.698	1.26	5.70	1.38	2.51	1+1.88a <sub>v</sub>	1+0.22a <sub>x</sub>
12 x 6a	54	15.879	.500	.883	1.33	4.86	1.42	2.52	1+1.69a <sub>v</sub>	1+0.26a <sub>x</sub>
12 x 6b	44	12.946	.400	.717	1.31	4.93	1.35	2.48	1+1.75a <sub>v</sub>	1+0.25a <sub>x</sub>
12 x 5	32	9.408	.350	.550	1.02	4.83	1.42	2.54	1+2.41a <sub>v</sub>	1+0.26a <sub>x</sub>
10 x 8	70	20.582	.600	.970	1.86	4.09	1.35	2.49	1+1.15a <sub>v</sub>	1+0.30a <sub>x</sub>
10 x 6	42	12.358	.400	.736	1.38	4.14	1.33	2.46	1+1.62a <sub>v</sub>	1+0.29a <sub>x</sub>
10 x 5	30	8.820	.360	.552	1.05	4.06	1.41	2.52	1+2.26a <sub>v</sub>	1+0.30a <sub>x</sub>
9 x 7	58	17.064	.550	.924	1.64	3.67	1.36	2.51	1+1.29a <sub>v</sub>	1+0.34a <sub>x</sub>

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

Each weight per foot is for the shaft only. Weight of base, etc., to be added.

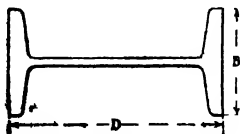
Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

W<sub>a</sub> = actual eccentric load; K = relative eccentricity coefficient; W<sub>e</sub> = equivalent concentric value; W<sub>e</sub> = W<sub>a</sub> x K.

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for a<sub>v</sub> and a<sub>x</sub> respectively.

For full explanations of tables, see notes commencing page 192.

# REDPATH, BROWN & CO., LIMITED.



## STANCHIONS.

### Steel Joists.

Safe Concentric Loads, in Tons.  
Ends Flat.

Reference Mark	Size, D x B inches	HEIGHTS IN FEET																
		3	4	5	6	7	8	9	10	11	12	13	14	15	16	17		
15 J	9 x 4	40	0	38	9	37	5	35	8	33	8	27	9	22	0	17	8	
14 J	8 x 6	68	0	67	3	66	4	65	3	64	0	62	4	60	7	58	8	56
13 J	8 x 5	54	1	53	4	52	3	51	1	49	6	47	9	46	0	13	5	36
12 J	8 x 4	34	2	33	3	32	1	30	6	28	9	23	7	18	7	15	2	
11 J	7 x 4	30	5	29	7	28	7	27	5	26	1	22	6	17	9	14	5	12
10 J	6 x 5	48	3	47	6	46	7	45	6	44	2	42	7	41	0	38	7	32
9 J	6 x 4	38	4	37	6	36	6	35	4	34	0	32	4	28	4	23	0	19
8 J	6 x 3	22	2	21	1	19	6	15	8	11	6	8	9					
7 J	5 x 4	34	7	34	1	33	3	32	4	31	3	30	0	28	6	24	0	19
6 J	5 x 3	20	6	19	7	18	6	17	3	12	7	9	7					
4 J	4 x 3	17	8	17	0	16	1	15	0	11	1	8	5	6	7			
2 J	3 x 3	16	0	15	4	14	6	13	7	10	9	8	4	6	6			
1 J	3 x 1½	5	8	3	3													

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160  
Safe loads are calculated by the Moncreff Formulae for stanchions of mild steel having "both ends flat"

Safe loads for the condition of "both ends fixed" are identical with tabular loads on the heights to left of zigzag line

For other conditions and formulae, see notes commencing page 192.

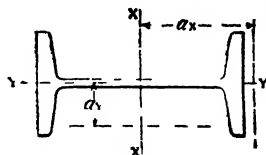
For explanations of properties, &c, see Part IV

# REDPATH, BROWN & CO., LIMITED.

## STANCHIONS.

### Steel Joists.

#### Dimensions and Properties.



Size, D x B inches.	Weight per foot in lbs	Area in square inches	Standard Thicknesses.		Radii of Gyration.		Eccentricity Coefficients.			
			Web	Flange.	Axis Y-Y	Axis X-X	Web.	Flange.	Axis Y-Y	Axis X-X
9 x 4	21	6.178	.300	.460	0.82	3.62	1.44	2.54	$1 + 2.95a_v$	$1 + 0.34a_x$
8 x 6	35	10.293	.440	.597	1.32	3.28	1.38	2.49	$1 + 1.72a_v$	$1 + 0.37a_x$
8 x 5	28	8.211	.350	.575	1.11	3.29	1.35	2.48	$1 + 2.01a_v$	$1 + 0.37a_x$
8 x 4	18	5.297	.280	.402	0.82	3.24	1.42	2.52	$1 + 2.97a_v$	$1 + 0.38a_x$
7 x 4	16	4.709	.250	.387	0.85	2.88	1.35	2.47	$1 + 2.76a_v$	$1 + 0.42a_x$
6 x 5	25	7.354	.410	.520	1.11	2.43	1.42	2.52	$1 + 2.02a_v$	$1 + 0.51a_x$
6 x 4½	20	5.882	.370	.431	0.96	2.42	1.45	2.53	$1 + 2.45a_v$	$1 + 0.51a_x$
6 x 3	12	3.527	.260	.348	0.61	2.39	1.52	2.57	$1 + 3.96a_v$	$1 + 0.53a_x$
5 x 4½	18	5.290	.290	.448	1.03	2.07	1.31	2.46	$1 + 2.11a_v$	$1 + 0.58a_x$
5 x 3	11	3.238	.220	.376	0.67	2.05	1.37	2.49	$1 + 3.32a_v$	$1 + 0.60a_x$
4 x 3	9½	2.795	.220	.336	0.67	1.64	1.36	2.49	$1 + 3.28a_v$	$1 + 0.74a_x$
3 x 3	8½	2.501	.200	.332	0.71	1.23	1.30	2.49	$1 + 2.98a_v$	$1 + 0.99a_x$
3 x 1½	4	1.176	.160	.248	0.32	1.18	1.57	2.60	$1 + 7.10a_v$	$1 + 1.07a_x$

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

Each weight per foot is for the shaft only. Weight of base, &c., to be added.

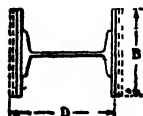
Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

We=actual eccentric load; K=relative eccentricity coefficient; Wc=equivalent concentric load; Wc=WexK.

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for a<sub>v</sub> and a<sub>x</sub> respectively.

For full explanations of tables, see notes commencing page 192.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND STANCHIONS.

Safe Concentric Loads, in Tons.  
Ends Flat.

Reference Mark.	Size, D x B inches.	HEIGHTS IN FEET.															
		10	12	14	16	18	20	22	24	26	28	30	32	36	40		
278 K	27 x 14	466	461	455	448	440	432	422	411	400	388	355	312	247	200		
276 K	26½ x "	420	415	409	403	395	387	377	367	356	345	302	265	209	169		
274 K	26 x "	374	369	363	357	349	341	333	323	313	286	249	219	173	140		
273 K	25½ x "	350	345	340	333	326	318	309	300	290	273	221	194	163			
272 K	25¼ x "	327	322	317	310	303	295	286	277	258	222	193	170	134			
271 K	25½ x "	303	298	293	286	279	271	262	253	222	191	166	146	115			
258 K	23 x 14	446	441	435	429	422	414	405	395	385	374	354	311	245	199		
256 K	22½ x "	399	395	390	384	377	369	361	351	342	331	299	263	208	168		
254 K	22 x "	353	349	344	338	331	324	316	307	298	282	245	216	170	138		
253 K	21½ x "	330	325	321	315	308	301	293	285	276	271	219	192	152	123		
252 K	21¼ x "	306	302	297	292	285	278	270	262	253	220	191	168	133			
251 K	21½ x "	283	279	274	268	262	255	247	239	219	189	161	145	114			
238 K	21 x 12	377	371	365	358	350	340	331	320	301	260	226	199	157			
236 K	20½ x "	337	332	326	319	311	303	294	283	256	220	192	169	133			
234 K	20 x "	297	293	287	280	273	265	256	247	210	181	158	139				
233 K	19½ x "	278	273	267	261	254	246	237	220	188	162	141	124				
232 K	19¼ x "	258	253	248	241	234	226	218	194	165	142	121	109				
231 K	19½ x "	238	233	228	221	214	207	198	167	142	123	107	94½				

Rivets ¾ in diam at 6 in pitch

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160.

Safe loads are calculated by the Moncrieff Formulae for stanchions of mild steel having "both ends flat."

Safe loads for the condition of "both ends fixed" are identical with tabular loads on the heights to left of zigzag line

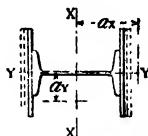
For other conditions and formulae, see notes commencing page 192

For explanations of properties, &c, see Part IV

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND STANCHIONS.

### Composition and Properties.



Composed of		Weight per foot in lbs	Area in square inches.	Radii of Gyration.		Eccentricity Coefficients.			
One Steel Joist.	Plates, each flange to form.			Axis Y-Y	Axis X-X	Web.	Flange.	Axis Y-Y	Axis X-X
24 x 7½	14 x 1½	246½	71.4	3.24	11.53	1.20	2.37	1+0.67a <sub>v</sub>	1+0.10a <sub>x</sub>
"	" x 1½	223	64.4	3.15	11.31	1.21	2.37	1+0.71a <sub>v</sub>	1+0.11a <sub>x</sub>
"	" x 1	199	57.4	3.03	11.07	1.23	2.38	1+0.76a <sub>v</sub>	1+0.11a <sub>x</sub>
"	" x ¾	187	53.9	2.94	10.94	1.24	2.39	1+0.81a <sub>v</sub>	1+0.11a <sub>x</sub>
"	" x ¾	175	50.4	2.85	10.79	1.26	2.40	1+0.86a <sub>v</sub>	1+0.11a <sub>x</sub>
"	" x ¾	163½	46.9	2.74	10.64	1.28	2.41	1+0.93a <sub>v</sub>	1+0.11a <sub>x</sub>
20 x 7½	14 x 1½	235½	68.1	3.31	9.79	1.19	2.38	1+0.64a <sub>v</sub>	1+0.12a <sub>x</sub>
"	" x 1½	212	61.1	3.22	9.59	1.20	2.38	1+0.68a <sub>v</sub>	1+0.12a <sub>x</sub>
"	" x 1	188	54.1	3.10	9.37	1.22	2.38	1+0.73a <sub>v</sub>	1+0.13a <sub>x</sub>
"	" x ¾	179½	50.6	3.02	9.25	1.23	2.38	1+0.77a <sub>v</sub>	1+0.13a <sub>x</sub>
"	" x ¾	164	47.1	2.93	9.13	1.25	2.39	1+0.82a <sub>v</sub>	1+0.13a <sub>x</sub>
"	" x ¾	152½	43.6	2.82	8.99	1.27	2.40	1+0.88a <sub>v</sub>	1+0.13a <sub>x</sub>
18 x 7	12 x 1½	201	58.0	2.87	8.88	1.20	2.40	1+0.78a <sub>v</sub>	1+0.14a <sub>x</sub>
"	" x 1½	181	52.0	2.79	8.69	1.21	2.39	1+0.77a <sub>v</sub>	1+0.14a <sub>x</sub>
"	" x 1	160½	46.0	2.69	8.48	1.23	2.39	1+0.83a <sub>v</sub>	1+0.14a <sub>x</sub>
"	" x ¾	150	43.0	2.63	8.37	1.24	2.39	1+0.87a <sub>v</sub>	1+0.14a <sub>x</sub>
"	" x ¾	140	40.0	2.56	8.26	1.25	2.40	1+0.92a <sub>v</sub>	1+0.15a <sub>x</sub>
"	" x ¾	130	37.0	2.47	8.13	1.27	2.40	1+0.98a <sub>v</sub>	1+0.16a <sub>x</sub>

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

Each weight per foot is for the riveted shaft only. Weight of base, &c., to be added.

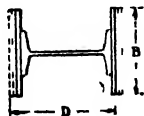
Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

We=actual eccentric load; K=relative eccentricity coefficient; Wc=equivalent concentric value; Wc=We x K.

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for  $a_v$  and  $a_x$  respectively.

For full explanations of tables, see notes commencing page 192.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND STANCHIONS.

Safe Concentric Loads, in Tons.  
Ends Flat.

Reference Mark.	Size, D x B inches	HEIGHTS IN FEET.															
		10	12	14	16	18	20	22	24	26	28	30	32	34	36		
218 K	19 x 12	352	347	342	335	327	319	310	300	289	249	217	191	169	150		
216 K	18½ x "	313	308	303	296	289	282	273	264	243	210	183	161	142	127		
214 K	18 x "	273	269	264	258	251	244	236	227	198	171	149	131	116	103		
213 K	17½ x "	253	249	244	238	232	225	217	206	175	151	132	116	102			
212 K	17½ x "	233	229	224	219	212	206	198	179	153	132	115	101	89.4			
211 K	17½ x "	213	209	204	199	193	186	179	153	130	112	97.8	85.9				
210 K	17 x "	193	189	184	179	173	166	150	126	107	92.8	80.8					
198 K	18 x 12	347	342	336	330	323	315	306	296	286	249	217	191	169	158		
196 K	17½ x "	307	303	298	292	285	277	269	260	244	210	183	161	143	127		
194 K	17 x "	267	263	259	253	247	240	232	224	199	171	149	131	116	104		
193 K	16½ x "	248	244	239	234	227	221	213	206	176	152	132	116	103	91.9		
192 K	16½ x "	228	224	219	214	208	202	195	180	153	132	115	101	89.8			
191 K	16½ x "	208	204	200	194	189	182	175	154	131	113	98.3	86.4				
190 K	16 x "	188	184	180	174	169	162	151	127	108	93.4	81.4	71.5				
178 K	17 x 12	343	338	333	327	320	312	303	294	284	249	217	191	169	151		
176 K	16½ x "	303	299	294	288	282	274	266	258	244	210	183	153	142	127		
174 K	16 x "	264	260	255	250	244	237	229	222	199	171	149	131	116	103		
173 K	15½ x "	244	240	236	230	224	218	211	203	176	152	132	116	103	91.8		
172 K	15½ x "	224	220	216	211	205	199	192	180	153	132	115	101	89.7			
171 K	15½ x "	204	201	196	191	186	179	173	153	131	113	98.2	86.3	76.4			
170 K	15 x "	184	181	176	171	166	160	151	127	108	93.2	81.1	71.3				

Rivets ¾ in diam. at 6 in. pitch

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160.

Safe loads are calculated by the Moncrieff Formulae for stanchions of mild steel having "both ends flat."

Safe loads for the condition of "both ends fixed" are identical with tabular loads on the heights to left of zigzag line.

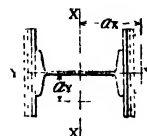
For other conditions and formulae, see notes commencing page 102.

For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND STANCHIONS.

### Composition and Properties.



Composed of		Weight per foot in lbs.	Area in square inches.	Radii of Gyration.		Eccentricity Coefficients.			
One Steel Joist.	Plates, each flange to form.			Axis Y-Y	Axis X-X	Web.	Flange.	Axis Y-Y	Axis X-X
16 x 6	12 x 1 1/4	187	54.2	2.91	8.02	1.20	2.41	1+0.71a <sub>v</sub>	1+0.15a <sub>x</sub>
"	" x 1 1/4	166 1/2	48.2	2.83	7.83	1.21	2.40	1+0.75a <sub>v</sub>	1+0.15a <sub>x</sub>
"	" x 1	146	42.2	2.73	7.63	1.22	2.39	1+0.81a <sub>v</sub>	1+0.16a <sub>x</sub>
"	" x 7/8	136	39.2	2.66	7.52	1.23	2.39	1+0.85a <sub>v</sub>	1+0.16a <sub>x</sub>
"	" x 3/4	125 1/2	36.2	2.59	7.41	1.25	2.40	1+0.90a <sub>v</sub>	1+0.16a <sub>x</sub>
"	" x 5/8	115 1/2	33.2	2.49	7.28	1.27	2.40	1+0.97a <sub>v</sub>	1+0.16a <sub>x</sub>
"	" x 1/2	105 1/2	30.2	2.38	7.14	1.29	2.42	1+1.06a <sub>v</sub>	1+0.17a <sub>x</sub>
15 x 6	12 x 1 1/4	184	53.3	2.93	7.60	1.18	2.40	1+0.70a <sub>v</sub>	1+0.16a <sub>x</sub>
"	" x 1 1/4	163 1/2	47.3	2.86	7.43	1.18	2.39	1+0.73a <sub>v</sub>	1+0.16a <sub>x</sub>
"	" x 1	143	41.3	2.76	7.24	1.20	2.38	1+0.79a <sub>v</sub>	1+0.16a <sub>x</sub>
"	" x 7/8	133	38.3	2.70	7.13	1.21	2.38	1+0.82a <sub>v</sub>	1+0.17a <sub>x</sub>
"	" x 3/4	122 1/2	35.3	2.63	7.03	1.22	2.38	1+0.87a <sub>v</sub>	1+0.17a <sub>x</sub>
"	" x 5/8	112 1/2	32.3	2.53	6.91	1.24	2.38	1+0.93a <sub>v</sub>	1+0.17a <sub>x</sub>
"	" x 1/2	102 1/2	29.3	2.42	6.78	1.26	2.39	1+1.02a <sub>v</sub>	1+0.18a <sub>x</sub>
14 x 6a	12 x 1 1/4	182	52.7	2.95	7.15	1.17	2.41	1+0.69a <sub>v</sub>	1+0.17a <sub>x</sub>
"	" x 1 1/4	161 1/2	46.7	2.88	6.98	1.18	2.40	1+0.73a <sub>v</sub>	1+0.17a <sub>x</sub>
"	" x 1	141	40.7	2.78	6.80	1.20	2.39	1+0.78a <sub>v</sub>	1+0.18a <sub>x</sub>
"	" x 7/8	131	37.7	2.72	6.70	1.20	2.38	1+0.81a <sub>v</sub>	1+0.18a <sub>x</sub>
"	" x 3/4	120 1/2	34.7	2.65	6.59	1.22	2.38	1+0.86a <sub>v</sub>	1+0.18a <sub>x</sub>
"	" x 5/8	110 1/2	31.7	2.56	6.48	1.23	2.38	1+0.92a <sub>v</sub>	1+0.18a <sub>x</sub>
"	" x 1/2	100 1/2	28.7	2.44	6.36	1.25	2.39	1+1.01a <sub>v</sub>	1+0.19a <sub>x</sub>

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2 1/2 per cent. over this must be allowed. See page 7.

Each weight per foot is for the riveted shaft only. Weight of base, &c., to be added.

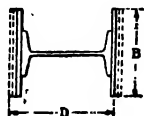
Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

W<sub>e</sub>=actual eccentric load; K=relative eccentricity coefficient; W<sub>c</sub>=equivalent concentric value; W<sub>c</sub>=W<sub>e</sub> x K.

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for a<sub>v</sub> and a<sub>x</sub> respectively.

For full explanations of tables, see notes commencing page 192.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND STANCHIONS.

Safe Concentric Loads, in Tons.  
Ends Flat.

Reference Mark.	Size, D x B inches.	HEIGHTS IN FEET															
		10	12	14	16	18	20	22	24	26	28	30	32	34	36		
158 K	17 x 12	322	318	313	308	302	295	287	278	270	246	214	188	167	149		
156 K	16½ x "	283	279	275	270	264	257	250	243	235	207	180	158	140	125		
154 K	16 x "	243	240	236	231	226	220	214	207	195	168	146	128	114	101		
153 K	15½ x "	224	220	216	212	207	201	195	188	172	148	129	113	100	89.7		
152 K	15 x "	204	201	197	193	188	182	176	170	149	129	112	98.6	87.4	77.9		
151 K	15½ x "	184	181	177	173	168	163	158	149	127	109	95.2	83.7	74.1			
150 K	15 x "	164	161	158	154	149	144	139	122	104	89.8	78.3	68.8				
138 K	15 x 12	337	333	328	322	315	307	299	290	280	249	217	191	169	151		
136 K	14½ x "	298	294	289	283	277	270	262	254	244	211	183	161	143	127		
134 K	14 x "	258	254	250	245	239	232	225	218	199	171	149	131	116	104		
133 K	13½ x "	238	235	230	225	220	214	207	199	176	152	132	116	103	91.9		
132 K	13 x "	219	215	211	206	201	195	188	180	153	132	115	101	89.8			
131 K	13½ x "	199	195	191	187	181	175	169	154	131	113	98.4	86.5	76.6			
130 K	13 x "	179	175	171	167	162	156	150	127	108	93.4	81.4	71.5				
118 K	15 x 12	319	315	310	304	298	291	284	276	267	246	214	188	167	149		
116 K	14½ x "	279	276	271	266	261	254	248	240	232	207	180	159	140	125		
114 K	14 x "	240	236	232	228	223	217	211	204	195	168	146	129	114	102		
113 K	13½ x "	220	217	213	209	204	198	192	186	172	149	129	114	101	90.0		
112 K	13 x "	200	197	194	189	185	180	174	168	150	129	112	98.9	87.6	78.2		
111 K	13½ x "	180	177	174	170	166	161	155	149	127	109	95.5	83.9	74.4			
110 K	13 x "	160	158	154	150	146	141	136	123	101	90.2	78.6	69.0	61.1			

Rivets ¾-in. diam. at 6-in. pitch.

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160.

Safe loads are calculated by the Moncrieff Formulæ for stanchions of mild steel having "both ends flat."

Safe loads for the condition of "both ends fixed" are identical with tabular loads on the heights to left of zigzag line.

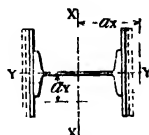
For other conditions and formulæ, see notes commencing page 192.

For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND STANCHIONS.

### Composition and Properties.



Composed of		Weight per foot in lbs.	Area in square inches.	Radii of Gyration.		Eccentricity Coefficients.			
One Steel Joist.	Plates, each flange to form.			Axis Y-Y	Axis X-X	Web.	Flange.	Axis Y-Y	Axis X-X
14 x 6b	12 x 1 1/2	171	49.5	3.02	7.26	1.13	2.37	1 + 0.66a <sub>v</sub>	1 + 0.16ax
"	" x 1 1/2	150 1/2	43.5	2.96	7.09	1.14	2.36	1 + 0.69a <sub>v</sub>	1 + 0.17ax
"	" x 1	130	37.5	2.87	6.91	1.15	2.34	1 + 0.73a <sub>v</sub>	1 + 0.17ax
"	" x 7/8	120	34.5	2.81	6.81	1.15	2.34	1 + 0.76a <sub>v</sub>	1 + 0.17ax
"	" x 3/4	109 1/2	31.5	2.74	6.71	1.16	2.34	1 + 0.80a <sub>v</sub>	1 + 0.17ax
"	" x 5/8	99 1/2	28.5	2.66	6.60	1.17	2.34	1 + 0.85a <sub>v</sub>	1 + 0.18ax
"	" x 1/2	89 1/2	25.5	2.54	6.48	1.19	2.34	1 + 0.93a <sub>v</sub>	1 + 0.18ax
12 x 6a	12 x 1 1/2	179	51.8	2.98	6.24	1.17	2.45	1 + 0.68a <sub>v</sub>	1 + 0.19ax
"	" x 1 1/2	158 1/2	45.8	2.91	6.08	1.18	2.42	1 + 0.71a <sub>v</sub>	1 + 0.20ax
"	" x 1	138	39.8	2.81	5.91	1.19	2.41	1 + 0.76a <sub>v</sub>	1 + 0.20ax
"	" x 7/8	128	36.8	2.75	5.81	1.20	2.40	1 + 0.79a <sub>v</sub>	1 + 0.21ax
"	" x 3/4	117 1/2	33.8	2.68	5.72	1.21	2.40	1 + 0.83a <sub>v</sub>	1 + 0.21ax
"	" x 5/8	107 1/2	30.8	2.59	5.61	1.22	2.39	1 + 0.89a <sub>v</sub>	1 + 0.21ax
"	" x 1/2	97 1/2	27.8	2.48	5.50	1.24	2.40	1 + 0.97a <sub>v</sub>	1 + 0.22ax
12 x 6b	12 x 1 1/2	169	48.9	3.04	6.33	1.13	2.41	1 + 0.65a <sub>v</sub>	1 + 0.19ax
"	" x 1 1/2	148 1/2	42.9	2.98	6.17	1.14	2.38	1 + 0.68a <sub>v</sub>	1 + 0.19ax
"	" x 1	128	36.9	2.90	6.00	1.14	2.36	1 + 0.72a <sub>v</sub>	1 + 0.20ax
"	" x 7/8	118	33.9	2.84	5.91	1.15	2.35	1 + 0.74a <sub>v</sub>	1 + 0.20ax
"	" x 3/4	107 1/2	30.9	2.77	5.82	1.16	2.35	1 + 0.78a <sub>v</sub>	1 + 0.20ax
"	" x 5/8	97 1/2	27.9	2.69	5.72	1.17	2.34	1 + 0.83a <sub>v</sub>	1 + 0.20ax
"	" x 1/2	87 1/2	24.9	2.58	5.61	1.18	2.35	1 + 0.90a <sub>v</sub>	1 + 0.21ax

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2 1/2 per cent. over this must be allowed. See page 7.

Each weight per foot is for the riveted shaft only. Weight of base, &c., to be added.

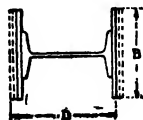
Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

We=actual eccentric load; K=relative eccentricity coefficient; Wc=equivalent concentric value; Wc=We x K.

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for a, and ax respectively.

For full explanations of tables, see notes commencing page 192.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND STANCHIONS.

Safe Concentric Loads, in Tons.  
Ends Flat.

Reference Mark.	Size, D x B inches.	HEIGHTS IN FEET.															
		10	12	14	16	18	20	22	24	26	28	30	32	34	36		
102 K	13½ × 10	156	152	148	144	139	133	118	99.4	84.7	77.3	63.6					
101 K	13½ × "	139	136	132	128	123	118	100	84.0	71.6	61.7	53.8					
100 K	13 × "	123	120	116	112	107	98.9	81.8	68.7	58.5	50.5						
99 K	12½ × "	106	103	99.9	95.9	91.5	76.7	63.4	53.3	45.4							
92 K	14 × 14	503	498	493	487	480	472	463	454	444	433	422	400	363	323		
90 K	13½ × "	456	452	447	441	435	428	420	411	402	392	381	362	321	286		
88 K	13 × "	410	406	402	396	390	383	376	368	359	350	340	314	278	248		
86 K	12½ × 12	328	324	319	313	306	298	290	281	271	264	204	179	159	142		
84 K	12 × "	289	285	280	274	268	261	253	244	226	195	170	149	132	118		
83 K	11½ × "	269	265	260	255	249	242	234	226	203	175	153	134	119	106		
82 K	11½ × "	249	245	241	235	229	223	216	208	181	156	136	119	106	94.3		
81 K	11¼ × "	230	226	221	216	210	204	197	185	158	136	119	104	92.5			
80 K	11 × "	210	206	201	196	191	184	178	159	136	117	102	89.6	79.3			
68 K	13 × 12	315	311	306	301	295	288	281	273	265	247	215	189	167	149		
66 K	12½ × "	275	272	268	263	257	251	245	238	230	208	181	159	141	125		
64 K	12 × "	236	233	229	225	220	214	208	202	195	168	147	129	114	102		
63 K	11½ × "	216	213	210	205	201	195	190	184	173	149	130	114	101	90.2		
62 K	11¼ × "	197	194	190	186	182	177	171	165	150	129	113	99.2	87.8	78.3		
61 K	11¼ × "	177	174	171	167	163	158	153	147	127	110	95.8	84.2	74.6	66.5		
60 K	11 × 10	141	138	134	129	124	113	93.2	78.3	66.7	57.6						
59 K	10½ × "	125	121	117	113	107	90.8	75.0	63.0	53.7							

Rivets ¾-in. diam. at 6-in. pitch.

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160.

Safe loads are calculated by the Moncrieff Formulae for stanchions of mild steel having "both ends flat."

Safe loads for the condition of "both ends fixed" are identical with tabular loads on the heights to left of zigzag line.

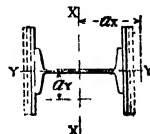
For other conditions and formulae, see notes commencing page 192.

For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND STANCHIONS.

### Composition and Properties.



Composed of		Weight per foot in lbs.	Area in square inches.	Radii of Gyration.		Eccentricity Coefficients.			
One Steel Joist.	Plates, each flange to form.			Axis Y-Y	Axis X-X	Web.	Flange.	Axis Y-Y	Axis X-X
12 x 5	10 x $\frac{3}{8}$	85 $\frac{1}{2}$	24.4	2.35	5.83	1.16	2.34	1+0.91a <sub>v</sub>	1+0.20a <sub>x</sub>
"	" x $\frac{1}{2}$	77	21.9	2.28	5.72	1.17	2.34	1+0.96a <sub>v</sub>	1+0.20a <sub>x</sub>
"	" x $\frac{3}{8}$	68 $\frac{1}{2}$	19.4	2.19	5.60	1.18	2.35	1+1.04a <sub>v</sub>	1+0.21a <sub>x</sub>
"	" x $\frac{1}{2}$	60	16.9	2.06	5.47	1.21	2.36	1+1.17a <sub>v</sub>	1+0.21a <sub>x</sub>
10 x 8	14 x 2	264	76.6	3.59	5.57	1.17	2.58	1+0.55a <sub>v</sub>	1+0.23a <sub>x</sub>
"	" x 1 $\frac{1}{2}$	240 $\frac{1}{2}$	69.6	3.54	5.42	1.17	2.55	1+0.56a <sub>v</sub>	1+0.23a <sub>x</sub>
"	" x 1 $\frac{1}{2}$	216 $\frac{1}{2}$	62.6	3.48	5.27	1.18	2.52	1+0.58a <sub>v</sub>	1+0.24a <sub>x</sub>
"	12 x 1 $\frac{1}{2}$	176	50.6	2.92	5.06	1.21	2.52	1+0.71a <sub>v</sub>	1+0.25a <sub>x</sub>
"	" x 1	155 $\frac{1}{2}$	44.6	2.84	4.90	1.23	2.50	1+0.75a <sub>v</sub>	1+0.25a <sub>x</sub>
"	" x $\frac{7}{8}$	145	41.6	2.79	4.82	1.23	2.49	1+0.77a <sub>v</sub>	1+0.25a <sub>x</sub>
"	" x $\frac{3}{4}$	135	38.6	2.73	4.73	1.24	2.48	1+0.81a <sub>v</sub>	1+0.26a <sub>x</sub>
"	" x $\frac{1}{2}$	125	35.6	2.66	4.65	1.26	2.47	1+0.85a <sub>v</sub>	1+0.26a <sub>x</sub>
"	" x $\frac{3}{8}$	114 $\frac{1}{2}$	32.6	2.57	4.55	1.27	2.46	1+0.91a <sub>v</sub>	1+0.27a <sub>x</sub>
10 x 6	12 x 1 $\frac{1}{2}$	167	48.3	3.06	5.39	1.13	2.45	1+0.64a <sub>v</sub>	1+0.23a <sub>x</sub>
"	" x 1 $\frac{1}{2}$	146 $\frac{1}{2}$	42.3	3.00	5.24	1.13	2.42	1+0.67a <sub>v</sub>	1+0.23a <sub>x</sub>
"	" x 1	126	36.3	2.92	5.08	1.14	2.40	1+0.70a <sub>v</sub>	1+0.23a <sub>x</sub>
"	" x $\frac{3}{4}$	116	33.3	2.87	5.00	1.15	2.38	1+0.73a <sub>v</sub>	1+0.24a <sub>x</sub>
"	" x $\frac{1}{2}$	105 $\frac{1}{2}$	30.3	2.80	4.91	1.15	2.37	1+0.76a <sub>v</sub>	1+0.24a <sub>x</sub>
"	" x $\frac{3}{8}$	95 $\frac{1}{2}$	27.3	2.72	4.82	1.16	2.36	1+0.81a <sub>v</sub>	1+0.24a <sub>x</sub>
"	10 x $\frac{3}{4}$	78 $\frac{1}{2}$	22.3	2.18	4.67	1.21	2.39	1+1.06a <sub>v</sub>	1+0.25a <sub>x</sub>
"	" x $\frac{1}{2}$	70	19.8	2.07	4.56	1.23	2.39	1+1.16a <sub>v</sub>	1+0.26a <sub>x</sub>

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2 $\frac{1}{2}$  per cent. over this must be allowed. See page 7.

Each weight per foot is for the riveted shaft only. Weight of base, &c., to be added.

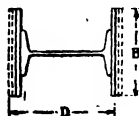
Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

We=actual eccentric load; K=relative eccentricity coefficient; Wc=equivalent concentric value; Wc=WxK.

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for a<sub>v</sub> and a<sub>x</sub> respectively.

For full explanations of tables, see notes commencing page 191.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND STANCHIONS.

Safe Concentric Loads, in Tons.  
Ends Flat.

Reference Mark.	Size, D × B inches.	HEIGHTS IN FEET.													
		8	10	12	14	16	18	20	22	24	26	28	30	32	34
54 K	12 × 10	188	185	181	177	172	167	161	154	130	111	95.7	83.4	73.3	
53 K	11½ × "	171	168	165	161	157	151	146	137	115	98.0	84.5	73.6	64.7	
52 K	11½ × "	155	152	149	145	141	136	131	118	99.5	84.8	73.1	63.7		
51 K	11½ × 9	129	126	123	118	114	108	91.0	75.3	63.2	53.9				
50 K	11 × "	114	111	108	104	99.7	92.5	74.9	61.9	52.0	44.3				
49 K	10½ × "	99.5	96.7	93.4	89.6	85.2	72.6	58.8	48.6	40.8					
38 K	12 × 12	349	345	341	335	329	322	315	307	297	288	259	226	198	176
36 K	11½ × "	309	306	302	297	291	285	278	270	262	253	220	192	169	149
34 K	11 × "	269	266	262	258	253	247	240	233	226	210	181	158	139	123
33 K	10½ × 10	224	221	216	210	204	197	189	169	142	121	104	90.7		
32 K	10½ × "	208	204	200	194	188	181	174	150	126	108	92.9	80.9		
31 K	10½ × "	191	188	183	178	172	166	159	132	111	94.6	81.6	71.1		
30 K	10 × "	175	171	167	162	156	150	138	114	95.6	81.5	70.2			
24 K	10 × 10	197	194	190	186	181	175	169	162	136	116	100	87.2	76.6	
23 K	9½ × "	181	178	174	170	165	160	154	144	121	103	88.8	77.3	68.0	
22 K	9½ × "	164	162	158	154	150	144	139	125	105	89.9	77.5	67.5		
21 K	9½ × 9	139	136	132	127	123	117	99.8	82.5	69.3	59.1				
20 K	9 × "	124	121	117	113	108	103	83.6	69.1	58.0	49.5				
19 K	8½ × "	109	106	103	98.8	94.2	83.3	67.5	55.8	46.8					
14 K	10 × 10	184	181	178	174	169	164	158	152	130	111	95.9	83.6	73.4	
13 K	9½ × "	168	165	162	158	154	149	143	137	115	98.1	84.6	73.7	64.8	
12 K	9½ × "	151	149	146	142	138	133	128	119	99.8	85.0	73.3	63.9	56.1	
11 K	9½ × 9	126	123	119	116	111	106	91.5	75.6	63.6	54.2	46.7			
10 K	9 × "	111	108	105	101	97.2	92.5	75.4	62.4	52.4	44.6				
9 K	8½ × "	96.0	93.5	90.4	86.9	82.9	73.3	59.3	49.0	41.2					
Rivets ¾-in diam. at 6-in. pitch.															

Rivets ¾-in. diam. at 6-in. pitch.

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160.

Safe loads are calculated by the Moseley's Formula for stanchions of mild steel having "both ends flat."

Safe loads for the condition of "both ends fixed" are identical with tabular loads on the heights to left of signag line.

For other conditions and formulae, see notes commencing page 192.

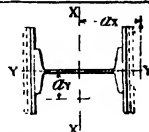
The safe load printed in italics is for a height greater than 20D.

For explanations of properties, &c., see Part IV

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND STANCHIONS.

### Composition and Properties.



Composed of		Weight per foot in lbs.	Area in square inches.	Radii of Gyration.		Eccentricity Coefficients.			
One Steel Joist.	Plates, each flange to form.			Axis Y-Y	Axis X-X	Web.	Flange.	Axis Y-Y	Axis X-X
10 x 5	10 x 1	100½	28.8	2.47	5.11	1.15	2.38	1+0.82ax	1+0.23ax
"	" x 7/8	92	26.3	2.43	5.02	1.15	2.37	1+0.85ax	1+0.23ax
"	" x 3/4	83½	23.8	2.38	4.93	1.16	2.36	1+0.89ax	1+0.24ax
"	9 x 3/4	70½	20.0	2.06	4.80	1.19	2.37	1+1.06ax	1+0.24ax
"	" x 3/8	63	17.8	1.99	4.70	1.21	2.37	1+1.14ax	1+0.25ax
"	" x 3/8	55½	15.5	1.88	4.58	1.23	2.38	1+1.27ax	1+0.26ax
9 x 7	12 x 1½	184	53.0	3.00	4.81	1.19	2.56	1+0.67ax	1+0.26ax
"	" x 1½	164	47.0	2.94	4.66	1.19	2.53	1+0.70ax	1+0.27ax
"	" x 1	143½	41.0	2.85	4.50	1.20	2.50	1+0.74ax	1+0.27ax
"	10 x 7/8	121½	34.5	2.85	4.36	1.25	2.52	1+0.90ax	1+0.28ax
"	" x 3/4	113	32.0	2.81	4.28	1.26	2.51	1+0.94ax	1+0.29ax
"	" x 3/4	104½	29.5	2.25	4.19	1.27	2.50	1+0.98ax	1+0.29ax
"	" x 3/8	96	27.0	2.19	4.10	1.29	2.49	1+1.05ax	1+0.30ax
8 x 6	10 x 1	105½	30.3	2.47	4.13	1.18	2.47	1+0.82ax	1+0.29ax
"	" x 7/8	97	27.8	2.42	4.05	1.19	2.45	1+0.85ax	1+0.30ax
"	" x 3/4	88½	25.3	2.37	3.97	1.20	2.43	1+0.89ax	1+0.30ax
"	9 x 3/4	75½	21.5	2.09	3.85	1.23	2.44	1+1.03ax	1+0.31ax
"	" x 3/8	68	19.3	2.02	3.76	1.24	2.43	1+1.11ax	1+0.32ax
"	" x 3/8	60½	17.0	1.93	3.66	1.27	2.43	1+1.21ax	1+0.33ax
8 x 5	10 x 1	98½	28.2	2.50	4.19	1.14	2.43	1+0.80ax	1+0.29ax
"	" x 7/8	90	25.7	2.46	4.11	1.15	2.41	1+0.83ax	1+0.29ax
"	" x 3/4	81½	23.2	2.41	4.03	1.15	2.39	1+0.86ax	1+0.29ax
"	9 x 3/4	69	19.5	2.10	3.91	1.18	2.40	1+1.02ax	1+0.30ax
"	" x 3/8	61	17.2	2.03	3.82	1.19	2.39	1+1.10ax	1+0.31ax
"	" x 3/8	53½	15.0	1.93	3.72	1.21	2.38	1+1.21ax	1+0.32ax

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

Each weight per foot is for the riveted shaft only. Weight of base, &c., to be added.

Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

We = actual eccentric load; K = relative eccentricity coefficient; We = equivalent concentric value; We = We x K.

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for  $a_x$  and  $a_y$  respectively.

For full explanations of tables, see notes commencing page 192.

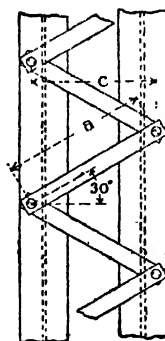
# REDPATH, BROWN & CO., LIMITED.



## COMPOUND STANCHIONS.

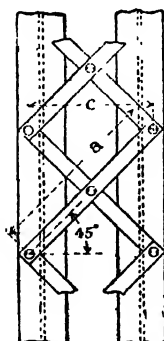
Safe Concentric Loads, in Tons.  
Ends Flat.

Reference Mark.	Size, D x B inches.	HEIGHTS IN FEET.													
		8	12	16	20	24	26	28	30	32	34	36	38	40	44
30 L	24 x 26	393	392	391	389	387	386	385	383	382	380	379	377	375	371
29 L	20 x 23	349	348	347	344	342	341	339	337	336	334	332	330	327	322
28 L	18 x 21	294	293	292	289	287	285	284	282	280	278	276	274	272	267
27 L	16 x 18	243	242	240	237	234	233	231	229	227	225	222	220	217	212
26 L	15 x 17	231	229	227	225	221	220	218	215	213	211	208	205	202	196
25 L	15 x 16	164	163	162	160	157	156	155	153	151	150	148	146	144	139
24 L	14 x 17	223	222	220	217	214	212	210	208	206	204	201	198	196	190
23 L	14 x 17	180	179	177	175	173	171	170	168	166	164	162	160	158	153

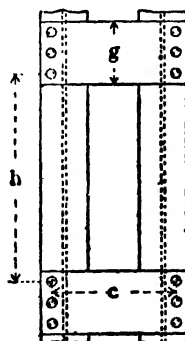


SINGLE LATTICING.

Suitable for values of  $c$ , not exceeding 15 inches.



DOUBLE LATTICING.



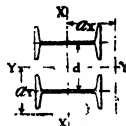
BATTEN PLATE.

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160.  
Safe loads are calculated by the Moncrieff Formula for stanchions of mild steel having "both ends flat."  
Safe loads for the condition of "both ends fixed" are identical with tabular loads for the above heights.  
For other conditions and formulae, see notes commencing page 192.  
For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND STANCHIONS.

### Composition and Properties.



Composed of two Steel Joists Latticed.	Weight per foot in lbs.	Area in square inches.	d. Centres of Webs, Inches.	Radii of Gyration.		Eccentricity Coefficients.			
				Axis Y-Y	Axis X-X	Web.	Flange.	Axis Y-Y	Axis X-X
24 x 7½	200	58.8	18.5	9.37	9.50	2.42	2.60	1+0.15a <sub>v</sub>	1+0.13a <sub>x</sub>
20 x 7½	178	52.3	15.5	7.90	7.99	2.48	2.57	1+0.19a <sub>v</sub>	1+0.16a <sub>x</sub>
18 x 7	150	44.1	14.0	7.15	7.22	2.50	2.56	1+0.21a <sub>v</sub>	1+0.17a <sub>x</sub>
16 x 6	124	36.4	12.0	6.12	6.31	2.51	2.61	1+0.24a <sub>v</sub>	1+0.20a <sub>x</sub>
15 x 6	118	34.7	11.0	5.64	6.02	2.53	2.55	1+0.27a <sub>v</sub>	1+0.21a <sub>x</sub>
15 x 5	84	24.7	11.0	5.59	5.89	2.47	2.62	1+0.28a <sub>v</sub>	1+0.22a <sub>x</sub>
14 x 6a	114	33.5	11.0	5.65	5.64	2.53	2.54	1+0.27a <sub>v</sub>	1+0.22a <sub>x</sub>
14 x 6b	92	27.0	11.0	5.64	5.70	2.52	2.51	1+0.27a <sub>v</sub>	1+0.22a <sub>x</sub>

CONVENTIONAL MAXIMUM SPACING AND MINIMUM PROPORTIONS OF LATTICE BARS AND BATTEN PLATES FOR CONCENTRIC LOADING (*Am. Ry. Engineering and Maintenance of Way Assoc.*).

Width of Joist Flange. Inches.	7½	7	6	5
Width of Lattice Bar. Inches.	2½	2½	2½	2½
Diameter of Rivet.	¾	¾	¾	¾

#### SINGLE LATTICING—

Maximum angle of inclination with horizontal = 30 degrees.

Minimum thickness = 1/40th of  $a$ , the diagonal centres of rivets.

Maximum horizontal centres of rivets,  $c$  = 15 inches.

#### DOUBLE LATTICING—

Maximum angle of inclination with horizontal = 45 degrees.

Minimum thickness = 1/60th of  $a$ , the diagonal centres of rivets.

#### BATTEN PLATES—

Maximum centres of end rivets of batten plates =  $h$  inches.

Let  $l$  = height of stanchion in inches, and  $k$  = radius of gyration of one joist.

Then  $h$  =  $l \times k$  least.

Then  $h$  =  $k$  greatest.

Minimum thickness = 1/60th of  $c$ , the horizontal centres of rivets.

Minimum width  $g$  =  $c$ , the horizontal centres of rivets for end plates.

" "  $g$  =  $\frac{1}{2}c$ , " " " " intermediate plates.

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

Each weight per foot is for the shaft only. Weights of lattices, batten, &c., to be added.

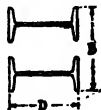
Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

$W_s$  = actual eccentric load;  $K$  = relative eccentricity coefficient;  $W_e$  = equivalent concentric value;  $W_o = W_s \times K$ .

In axial eccentricity coefficients substitute actual value of "term of eccentricity" for  $d_v$  and  $d_x$  respectively.

For full explanations of tables, see notes commencing page 102.

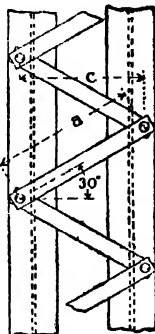
# REDPATH, BROWN & CO., LIMITED.



## COMPOUND STANCHIONS.

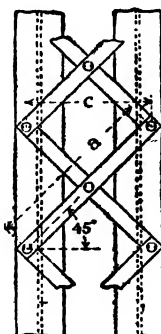
Safe Concentric Loads, in Tons.  
Ends Flat.

Reference Mark.	Size, D x B inches.	HEIGHTS IN FEET.															
		8	12	16	20	22	24	26	28	30	32	34	36	38	40		
22 L	12 x 15	211	209	206	203	201	198	196	193	190	187	184	181	177	173		
21 L	12 x 15	172	170	168	165	163	162	160	157	155	153	150	147	144	141		
20 L	12 x 14	125	124	122	120	118	117	116	114	112	110	108	106	104	102		
18 L	10 x 13	163	161	158	153	151	148	145	142	138	135	128	114	102	92.6		
17 L	10 x 12	116	115	112	109	107	105	103	100	97.9	95.3	86.7	77.3	69.4	62.6		
15 L	9 x 11	81.7	80.4	78.5	76.2	74.8	73.3	71.7	70.0	68.2	66.2	58.7	52.4	47.0	42.4		
14 L	8 x 12	136	133	129	125	122	119	116	112	106	91.3	81.3	72.5	65.1	58.7		
13 L	8 x 11	108	106	103	99.3	97.0	94.5	91.8	89.0	79.7	70.1	62.1	55.4	49.7	44.8		

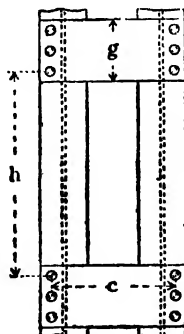


SINGLE LATTICING.

Suitable for values of  $c$ , not exceeding 15 inches.



DOUBLE LATTICING.



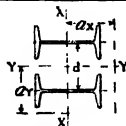
BATTEN PLATES.

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160.  
Safe loads are calculated by the Moncrieff Formula for stanchions of mild steel having "both ends flat."  
Safe loads for the condition of "both ends fixed" are identical with tabular loads on the heights to left of zigzag line.  
For other conditions and formulae, see notes commencing page 192.  
Safe loads printed in italics are for heights greater than 40D.  
For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND STANCHIONS.

### Composition and Properties.



Composed of two Steel Joists Latticed.	Weight per foot in lbs.	Area in square inches.	d. Centres of Webs. Inches.	Radii of Gyration.		Eccentricity Coefficients.			
				Axis Y-Y	Axis X-X	Web.	Flange.	Axis Y-Y	Axis X-X
12 x 6a	108	31.7	9.0	4.69	4.86	2.62	3.52	1+0.34a <sub>v</sub>	1+0.26a <sub>x</sub>
12 x 6b	88	25.9	9.0	4.68	4.93	2.61	2.48	1+0.34a <sub>v</sub>	1+0.25a <sub>x</sub>
12 x 5	64	18.8	9.0	4.61	4.83	2.54	2.54	1+0.33a <sub>v</sub>	1+0.26a <sub>x</sub>
10 x 6	84	24.7	7.0	3.75	4.14	2.71	2.46	1+0.46a <sub>v</sub>	1+0.29a <sub>x</sub>
10 x 5	60	17.6	7.0	3.65	4.06	2.65	2.52	1+0.45a <sub>v</sub>	1+0.30a <sub>x</sub>
9 x 4	42	12.3	7.0	3.59	3.62	2.55	2.54	1+0.43a <sub>v</sub>	1+0.34a <sub>x</sub>
8 x 6	70	20.6	6.0	3.27	3.23	2.80	2.49	1+0.56a <sub>v</sub>	1+0.37a <sub>x</sub>
8 x 5	56	16.5	6.0	3.20	3.29	2.71	2.48	1+0.54a <sub>v</sub>	1+0.37a <sub>x</sub>

CONVENTIONAL MAXIMUM SPACING AND MINIMUM PROPORTIONS OF LATTICE BARS AND BATTEN PLATES FOR CONCENTRIC LOADING (*Am. Ry. Engineering and Maintenance of Way Assoc.*).

Width of Joist Flange. Inches.	6	5	4
Width of Lattice Bar. Inches.	2½	2½	2
Diameter of Rivet.	¾	¾	¾

#### SINGLE LATTICING—

Maximum angle of inclination with horizontal = 30 degrees.

Minimum thickness = 1/40th of  $a$ , the diagonal centres of rivets.

Maximum horizontal centres of rivets,  $c$  = 15 inches.

#### DOUBLE LATTICING—

Maximum angle of inclination with horizontal = 45 degrees.

Minimum thickness = 1/80th of  $a$ , the diagonal centres of rivets.

#### BATTEN PLATES—

Maximum centres of end rivets of batten plates =  $h$  inches.

Let  $l$  = height of stanchion in inches, and  $k$  = radius of gyration of one joist.

$$l \times k \text{ least.}$$

Then  $h$  =  $k$  greatest.

Minimum thickness = 1/50th of  $c$ , the horizontal centres of rivets.

Minimum width  $g$  =  $c$ , the horizontal centres of rivets for end plates.

" "  $g$  = ¾, " " " intermediate plates.

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

Each weight per foot is for the shaft only. Weights of lattices, bases, &c., to be added.

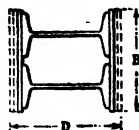
Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

$W_e$  = actual eccentric load;  $K$  = relative eccentricity coefficient;  $W_e$  = equivalent concentric value;  $W_e = W_e \times K$ .

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for  $a_v$  and  $a_x$  respectively.

For full explanations of tables, see notes commencing page 188.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND STANCHIONS

Safe Concentric Loads, in Tons.  
Ends Flat.

Reference Mark.	Size, D x B inches.	HEIGHTS IN FEET.															
		8	12	16	20	24	28	30	32	34	36	38	40	42	44		
282 M	28 x 24	1033	1028	1021	1013	1002	990	982	975	967	959	950	941	931	920		
280 M	27½ x "	952	948	942	934	924	912	906	899	892	884	876	867	858	848		
278 M	27 x "	872	868	863	855	846	836	829	823	816	809	801	794	785	776		
276 M	26½ x "	792	789	783	776	768	758	753	747	741	734	727	720	712	704		
274 M	26 x "	712	709	704	698	690	681	676	671	665	659	653	646	639	632		
273 M	25½ x 20	624	620	614	606	597	585	579	572	565	557	549	541	532	523		
272 M	25½ x "	591	587	581	574	565	554	548	541	534	527	520	512	503	495		
271 M	25½ x 18	540	536	529	521	511	498	491	484	476	469	460	451	442	434		
262 M	24 x 24	990	985	979	970	960	949	942	935	927	919	911	902	893	883		
260 M	23½ x "	909	905	899	892	883	872	865	859	852	845	837	828	820	811		
258 M	23 x "	829	825	820	813	805	795	789	783	777	770	763	755	747	739		
256 M	22½ x 20	681	677	670	662	652	640	633	626	618	610	602	593	584	574		
254 M	22 x "	615	610	605	597	588	577	571	564	557	550	542	534	525	517		
253 M	21½ x "	581	577	572	565	556	545	539	533	527	520	512	504	496	488		
252 M	21½ x "	548	544	539	532	524	514	508	502	496	489	482	475	467	452		
251 M	21½ x 18	498	493	487	480	470	459	453	446	439	432	424	416	408	393		
242 M	22 x 18	773	766	757	746	732	716	707	697	686	676	664	652	640	627		
240 M	21½ x "	713	707	699	688	675	660	651	642	633	623	612	601	589	577		
238 M	21 x "	653	647	640	630	618	604	596	588	579	570	560	550	539	528		
236 M	20½ x "	593	588	581	572	561	548	541	533	525	517	508	498	489	479		
234 M	20 x "	533	528	522	514	504	492	486	479	472	464	456	447	438	429		
233 M	19½ x 16	479	474	467	457	446	433	425	418	409	401	392	371	336	307		
232 M	19½ x "	452	447	441	432	421	408	402	394	386	378	370	348	316	288		
231 M	19½ x "	426	421	415	406	396	384	378	371	363	355	347	325	295	269		

Rivets ½-in. diam. at 6-in. pitch.

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160.

Safe loads are calculated by the Moncrieff Formulae for stanchions of mild steel having "both ends flat."

Safe loads for the condition of "both ends fixed" are identical with tabular loads on the heights to left of zigzag line.

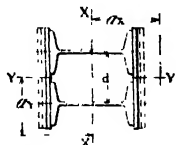
For other conditions and formulae, see notes commencing page 192.

For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND STANCHIONS.

### Composition and Properties.



Composed of		Weight per foot in lbs.	Area in square inches.	d. Centres of Webs in inches.	Radii of Gyration		Eccentricity Coefficients.			
Two Steel Joists.	Plates, each flange to form.				Axis Y-Y	Axis X-X	Web.	Flange.	Axis Y-Y	Axis X-X
24 x 7½	24 x 2	530	154.8	12	6.65	11.80	2.71	2.41	1+0.27ax	1+0.10ax
"	" x 1½	489½	142.8	"	6.63	11.61	2.72	2.40	1+0.27ax	1+0.10ax
"	" x 1½	448½	130.8	"	6.60	11.41	2.74	2.40	1+0.28ax	1+0.11ax
"	" x 1½	407½	118.8	"	6.57	11.19	2.75	2.40	1+0.28ax	1+0.11ax
"	" x 1	364	106.8	"	6.53	10.95	2.78	2.41	1+0.28ax	1+0.11ax
"	20 x ¾	322½	93.8	10	5.43	10.69	2.80	2.45	1+0.34ax	1+0.11ax
"	" x ¾	305½	88.8	"	5.41	10.56	2.81	2.46	1+0.34ax	1+0.12ax
"	18 x ¾	280	81.3	9	4.87	10.36	2.82	2.48	1+0.38ax	1+0.12ax
20 x 7½	24 x 2	508	148.3	12	6.68	10.05	2.70	2.43	1+0.27ax	1+0.12ax
"	" x 1½	467½	136.3	"	6.65	9.87	2.71	2.42	1+0.27ax	1+0.12ax
"	" x 1½	426½	124.3	"	6.63	9.69	2.72	2.41	1+0.27ax	1+0.12ax
"	20 x 1½	351½	102.3	10	5.50	9.37	2.75	2.44	1+0.33ax	1+0.13ax
"	" x 1	318	92.3	"	5.47	9.16	2.77	2.44	1+0.34ax	1+0.13ax
"	" x ¾	300½	87.3	"	5.45	9.05	2.78	2.45	1+0.34ax	1+0.13ax
"	" x ¾	283½	82.3	"	5.43	8.93	2.80	2.45	1+0.34ax	1+0.14ax
"	18 x ¾	258	74.8	9	4.89	8.75	2.81	2.47	1+0.38ax	1+0.14ax
18 x 7	18 x 2	398½	116.1	9	5.02	9.05	2.71	2.48	1+0.36ax	1+0.14ax
"	" x 1½	368	107.1	"	5.01	8.88	2.72	2.47	1+0.36ax	1+0.14ax
"	" x 1½	337½	98.1	"	4.99	8.71	2.73	2.46	1+0.36ax	1+0.14ax
"	" x 1½	307	89.1	"	4.97	8.52	2.74	2.45	1+0.37ax	1+0.14ax
"	" x 1	276½	80.1	"	4.94	8.32	2.76	2.45	1+0.37ax	1+0.15ax
"	16 x ¾	249	72.1	8	4.40	8.15	2.77	2.47	1+0.42ax	1+0.15ax
"	" x ¾	235½	68.1	"	4.38	8.04	2.78	2.47	1+0.42ax	1+0.15ax
"	" x ¾	222	64.1	"	4.37	7.93	2.79	2.47	1+0.42ax	1+0.16ax

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

Each weight per foot is for the riveted shaft only. Weight of base, &c., to be added.

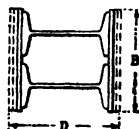
Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

We = actual eccentric load; K = relative eccentricity coefficient; We = equivalent concentric value; We = We x K.

In axial eccentricity coefficient is substitute actual value of "arm of eccentricity" for Gy and Gx respectively.

For full explanations of tables, see notes commencing page 182.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND STANCHIONS.

Safe Concentric Loads, in Tons.  
Ends Flat.

Reference Mark.	Size, D x B inches.	HEIGHTS IN FEET.															
		8	12	16	20	24	28	30	32	34	36	38	40	42	44		
218 M	19 x 16	561	555	547	536	523	508	499	490	481	471	460	441	400	365		
216 M	18½ x "	508	502	495	485	473	459	451	443	434	425	416	396	359	327		
214 M	18 x "	454	449	443	434	423	411	404	396	388	380	371	351	318	290		
213 M	17½ x 14	404	398	390	380	367	353	345	336	327	295	265	239	217	198		
212 M	17½ x "	380	375	367	358	346	332	325	317	308	277	248	224	203	185		
211 M	17½ x "	357	352	345	336	325	312	304	297	289	258	232	209	189	173		
210 M	17 x "	334	329	322	314	303	291	284	277	268	239	215	194	176	160		
198 M	18 x 16	549	543	535	525	512	497	489	480	471	461	451	434	394	359		
196 M	17½ x "	496	491	483	474	462	449	441	433	425	416	407	389	353	322		
194 M	17 x "	443	438	431	423	413	400	393	386	379	371	362	344	312	284		
193 M	16½ x 14	392	386	379	369	357	343	335	327	319	289	259	234	212	193		
192 M	16½ x "	369	364	356	347	336	322	315	307	299	270	242	219	198	181		
191 M	16½ x "	345	341	334	325	314	302	295	288	280	252	226	204	185	168		
190 M	16 x "	322	318	311	303	293	281	275	268	261	233	209	189	171	156		
184 M	17 x 14	349	344	337	328	318	305	298	291	283	257	231	208	189	172		
183 M	16½ x "	326	321	315	306	296	285	278	271	264	238	214	193	175	159		
182 M	16½ x "	302	298	292	284	275	264	258	252	245	219	197	178	161	147		
181 M	16½ x "	279	275	270	263	254	243	238	232	225	201	180	163	147	134		
180 M	16 x "	256	252	247	241	232	223	218	212	204	182	163	147	134	122		

Rivets ½-in. diam. at 6-in. pitch.

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160.

Safe loads are calculated by the Moncrieff Formulae for stanchions of mild steel having "both ends flat."

Safe loads for the condition of "both ends fixed" are identical with tabular loads on the heights to left of zigzag line.

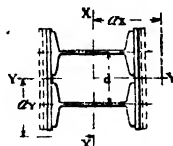
For other conditions and formulae, see notes commencing page 192.

For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND STANCHIONS.

### Composition and Properties.



Composed of		Weight per foot in lbs.	Area in square inches.	d. Centres of Webs in inches.	Radii of Gyration.		Eccentricity Coefficients.			
Two Steel Joists.	Plates, each flange to rim.				Axis Y-Y	Axis X-X	Web.	Flange.	Axis Y-Y	Axis X-X
16 x 6	16 x 1 1/2	289 1/2	84.4	8	4.43	7.80	2.74	2.49	1 + 0.41a <sub>v</sub>	1 + 0.16a <sub>x</sub>
"	" x 1 1/2	262 1/2	76.4	"	4.41	7.61	2.76	2.48	1 + 0.41a <sub>v</sub>	1 + 0.16a <sub>x</sub>
"	" x 1	235	68.4	"	4.39	7.42	2.78	2.47	1 + 0.42a <sub>v</sub>	1 + 0.17a <sub>x</sub>
"	14 x 7/8	210	60.9	7	3.84	7.24	2.79	2.50	1 + 0.48a <sub>v</sub>	1 + 0.17a <sub>x</sub>
"	" x 3/4	198	57.4	"	3.83	7.13	2.80	2.51	1 + 0.48a <sub>v</sub>	1 + 0.17a <sub>x</sub>
"	" x 3/8	186	53.9	"	3.82	7.02	2.81	2.51	1 + 0.48a <sub>v</sub>	1 + 0.18a <sub>x</sub>
"	" x 1/2	174	50.4	"	3.80	6.90	2.83	2.52	1 + 0.49a <sub>v</sub>	1 + 0.18a <sub>x</sub>
15 x 6	16 x 1 1/2	283 1/2	82.7	8	4.44	7.40	2.72	2.48	1 + 0.41a <sub>v</sub>	1 + 0.17a <sub>x</sub>
"	" x 1 1/2	256 1/2	74.7	"	4.43	7.23	2.74	2.47	1 + 0.41a <sub>v</sub>	1 + 0.17a <sub>x</sub>
"	" x 1	229 1/2	66.7	"	4.40	7.04	2.75	2.46	1 + 0.41a <sub>v</sub>	1 + 0.17a <sub>x</sub>
"	14 x 7/8	204	59.2	7	3.86	6.88	2.76	2.48	1 + 0.47a <sub>v</sub>	1 + 0.18a <sub>x</sub>
"	" x 3/4	192	55.7	"	3.84	6.78	2.78	2.48	1 + 0.48a <sub>v</sub>	1 + 0.18a <sub>x</sub>
"	" x 3/8	180	52.2	"	3.83	6.67	2.79	2.48	1 + 0.48a <sub>v</sub>	1 + 0.18a <sub>x</sub>
"	" x 1/2	168	48.7	"	3.82	6.56	2.80	2.49	1 + 0.48a <sub>v</sub>	1 + 0.19a <sub>x</sub>
15 x 5	14 x 1	181 1/2	52.7	7	3.85	7.09	2.75	2.44	1 + 0.47a <sub>v</sub>	1 + 0.17a <sub>x</sub>
"	" x 7/8	170	49.2	"	3.84	6.98	2.76	2.44	1 + 0.48a <sub>v</sub>	1 + 0.17a <sub>x</sub>
"	" x 3/4	158	45.7	"	3.82	6.87	2.78	2.44	1 + 0.48a <sub>v</sub>	1 + 0.18a <sub>x</sub>
"	" x 3/8	146	42.2	"	3.81	6.75	2.79	2.45	1 + 0.48a <sub>v</sub>	1 + 0.18a <sub>x</sub>
"	" x 1/2	134	38.7	"	3.78	6.62	2.81	2.46	1 + 0.49a <sub>v</sub>	1 + 0.18a <sub>x</sub>

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2 1/2 per cent. over this must be allowed. See page 7.

Each weight per foot is for the riveted shaft only. Weight of base, &c., to be added.

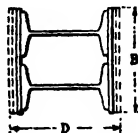
Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

W<sub>e</sub> = actual eccentric load; K = relative eccentricity coefficient; W<sub>c</sub> = equivalent concentric value; W<sub>c</sub> = W<sub>e</sub> × K.

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for a<sub>v</sub> and a<sub>x</sub> respectively.

For full explanations of tables, see notes commencing page 192.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND STANCHIONS.

Safe Concentric Loads, in Tons.  
Ends Flat.

Reference Mark.	Size, D x B inches.	HEIGHTS IN FEET.															
		8	12	16	20	24	28	30	32	34	36	38	40	42	44	46	48
168 M	17 x 16	541	536	528	518	505	491	482	474	465	455	445	429	389	357		
166 M	16½ x "	488	483	476	467	455	442	435	427	419	410	401	384	348	317		
164 M	16 x "	435	430	424	416	405	393	387	380	372	364	356	339	307	280		
163 M	15½ x 14	384	379	371	362	350	336	329	321	313	304	295	230	209	190		
162 M	15½ x "	361	356	349	340	329	316	309	301	293	285	278	215	195	178		
161 M	15½ x "	338	333	326	318	307	295	288	281	274	267	261	200	181	165		
160 M	15 x "	315	310	304	296	286	275	268	262	255	248	242	185	167	152		
148 M	17 x 16	498	493	486	477	465	452	445	437	428	420	411	398	361	329		
146 M	16½ x "	445	441	434	426	416	404	397	390	382	374	366	353	320	292		
144 M	16 x "	392	388	382	376	366	355	349	343	336	329	322	308	279	254		
143 M	15½ x 14	341	337	330	322	311	299	293	285	278	274	268	206	187	170		
142 M	15½ x "	318	314	308	300	290	279	272	266	259	255	251	191	173	157		
141 M	15½ x "	295	291	285	278	269	258	252	246	240	236	232	175	159	145		
140 M	15 x "	272	268	263	256	247	237	232	226	220	216	212	160	145	132		
128 M	15 x 14	488	482	473	461	446	429	420	410	400	371	333	301	273	248		
126 M	14½ x "	442	436	428	417	404	388	380	371	361	334	299	270	245	223		
124 M	14 x "	396	390	383	373	361	347	339	331	323	296	266	240	217	198		
123 M	13½ x "	372	367	360	351	340	326	319	311	303	277	249	224	204	185		
122 M	13½ x "	349	344	338	329	318	306	299	292	284	259	232	209	190	173		
121 M	13½ x "	326	322	315	307	297	285	279	272	265	240	215	194	176	160		
120 M	13 x "	303	299	293	285	276	265	259	252	246	221	198	179	162	148		

Rivets ½-in. diam. at 6-in. pitch.

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160.

Safe loads are calculated by the Moncrieff Formulae for stanchions of mild steel having "both ends flat."

Safe loads for the condition of "both ends fixed" are identical with tabular loads on the heights to left of zigzag line.

For other conditions and formulae, see notes commencing page 192.

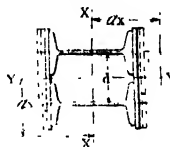
The safe load printed in italics is for a height greater than 40D.

For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND STANCHIONS.

### Composition and Properties.



Composed of		Weight per foot in lbs.	Area in square inches	d. Centres of Webs in inches.	Radii of Gyration.		Eccentricity Coefficients.			
Two Steel Jo.	Plates, each size in in.				Axis Y-Y	Axis X-X	Web.	Flange.	Axis Y-Y	Axis X-X
1 x 6a	16 x 1 1/2	279 1/2	61.5	8	4.45	6.97	2.72	2.49	1 + 0.41ax	1 + 0.18ax
"	" x 1 1/2	252 1/2	57.5	"	4.43	6.79	2.73	2.48	1 + 0.41ax	1 + 0.18ax
"	" x 1 1/2	225 1/2	53	"	4.41	6.61	2.75	2.46	1 + 0.41ax	1 + 0.18ax
"	14 x 1 1/2	200 1/2	50	7	3.86	6.49	2.76	2.49	1 + 0.47ax	1 + 0.19ax
"	" x 1 1/2	183	46.5	"	3.85	6.38	2.77	2.48	1 + 0.47ax	1 + 0.19ax
"	" x 1 1/2	176	45	"	3.84	6.26	2.79	2.48	1 + 0.48ax	1 + 0.20ax
"	" x 1 1/2	161	42	"	3.82	6.15	2.80	2.47	1 + 0.48ax	1 + 0.20ax
11 x 6a	16 x 1 1/2	257 1/2	57.0	8	4.47	7.09	2.63	2.41	1 + 0.40ax	1 + 0.17ax
"	" x 1 1/2	239 1/2	53.0	"	4.45	6.92	2.70	2.42	1 + 0.41ax	1 + 0.17ax
"	" x 1 1/2	203 1/2	50.0	"	4.43	6.71	2.72	2.41	1 + 0.41ax	1 + 0.18ax
"	14 x 1 1/2	178	46.5	7	3.87	6.59	2.73	2.43	1 + 0.47ax	1 + 0.18ax
"	" x 1 1/2	166	48.0	"	3.86	6.49	2.74	2.43	1 + 0.47ax	1 + 0.19ax
"	" x 1 1/2	154	44.5	"	3.85	6.38	2.75	2.43	1 + 0.47ax	1 + 0.19ax
"	" x 1 1/2	142	41.0	"	3.83	6.27	2.76	2.43	1 + 0.48ax	1 + 0.19ax
12 x 6a	14 x 1 1/2	253 1/2	73.7	7	3.91	6.02	2.71	2.55	1 + 0.46ax	1 + 0.21ax
"	" x 1 1/2	229 1/2	66.7	"	3.90	5.86	2.72	2.53	1 + 0.46ax	1 + 0.21ax
"	" x 1 1/2	205 1/2	59.7	"	3.88	5.69	2.74	2.51	1 + 0.47ax	1 + 0.22ax
"	" x 1 1/2	194	56.2	"	3.87	5.69	2.75	2.51	1 + 0.47ax	1 + 0.22ax
"	" x 1 1/2	182	52.7	"	3.86	5.51	2.76	2.50	1 + 0.47ax	1 + 0.22ax
"	" x 1 1/2	170	49.2	"	3.85	5.42	2.77	2.49	1 + 0.47ax	1 + 0.23ax
"	" x 1 1/2	158	45.7	"	3.84	5.32	2.78	2.49	1 + 0.48ax	1 + 0.23ax

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 24 per cent. over this must be allowed. See page 7.

Each weight per foot is for the riveted shaft only. Weight of base, &c., to be added.

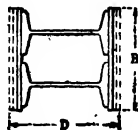
Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

We = actual eccentric load · K = relative eccentricity coefficient; We = equivalent concentric value; We = We · K.

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for  $a_x$  and  $a_y$  respectively.

For full explanations of tables, see notes commencing page 192.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND STANCHIONS.

Safe Concentric Loads, in Tons.  
Ends Flat.

Reference Mark.	Size, D x B inches.	HEIGHTS IN FEET.															
		8	12	16	20	24	28	30	32	34	36	38	40	42	44		
108 M	15 x 14	450	444	435	424	411	395	387	378	368	343	308	278	252	230		
106 M	14½ x "	408	398	390	380	368	354	347	338	330	306	275	248	225	205		
104 M	14 x "	357	352	345	336	326	313	306	299	291	263	241	217	197	180		
103 M	13½ x "	334	329	323	314	304	293	286	279	272	250	224	202	183	167		
102 M	13 x "	310	306	300	292	283	272	266	260	253	231	207	187	170	155		
101 M	13½ x "	287	283	278	270	262	251	246	240	234	212	190	172	156	142		
100 M	13 x "	264	260	255	248	240	231	226	220	214	193	174	157	142	129		
99 M	12½ x "	241	237	233	226	219	210	205	200	195	175	157	142	128	117		
94 M	14 x 12	282	277	270	260	249	235	225	198	175	156	140	127	115	105		
93 M	13½ x "	262	258	251	242	231	219	208	183	162	144	130	117	106	96.8		
92 M	13 x "	243	238	232	223	213	202	191	168	149	133	119	107	97.6	88.9		
91 M	13½ x "	223	219	213	205	196	185	174	153	136	121	108	98.0	88.9	81.0		
90 M	13 x "	203	199	194	187	178	168	157	138	122	109	98.0	88.5	80.2			
89 M	12½ x "	183	180	175	168	160	151	140	123	109	98.6	87.4	78.8	71.5			
78 M	13 x 14	442	436	428	417	404	389	381	372	362	339	304	275	249	227		
76 M	12½ x "	395	390	383	373	361	348	340	332	324	302	271	244	222	202		
74 M	12 x "	349	344	338	329	319	307	300	293	285	264	237	214	194	177		
73 M	11½ x "	326	321	315	307	297	286	280	273	266	245	220	199	180	164		
72 M	11 x "	303	298	293	285	276	265	260	253	247	227	203	184	166	152		
71 M	11½ x "	279	276	270	263	255	245	239	234	228	208	187	168	153	139		
70 M	11 x "	256	253	248	241	233	224	219	214	208	189	170	153	139	127		
69 M	10½ x "	233	230	225	219	212	204	199	194	189	170	153	138	125	114		

Rivets ¾-in. diam. at 6-in. pitch.

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160.

Safe loads are calculated by the Moncrieff Formulæ for stanchions of mild steel having "both ends flat."

Safe loads for the condition of "both ends fixed" are identical with tabular loads on the heights to left of zigzag line.

For other conditions and formulæ, see notes commencing page 192.

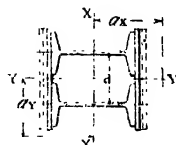
Safe loads printed in italics are for heights greater than 40D or B.

For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND STANCHIONS.

### Composition and Properties.



Composed of		Weight per foot in lbs.	Area in square inches.	d. Centres of Webs in inches.	Radii of Gyration		Eccentricity coefficients.			
Two Steel Joists.	Plates, each flange to form.				Axis Y-Y	Axis X-X	Web.	Flange.	Axis Y-Y	Axis X-X
12 x 6b	14 x 1 1/2	233 1/2	67.9	7	3.93	6.13	2.68	2.50	1 + 0.46a <sub>v</sub>	1 + 0.20a <sub>x</sub>
"	" x 1 1/2	209 1/2	60.9	"	3.91	5.97	2.69	2.48	1 + 0.46a <sub>v</sub>	1 + 0.21a <sub>x</sub>
"	" x 1	185 1/2	53.9	"	3.90	5.80	2.71	2.46	1 + 0.46a <sub>v</sub>	1 + 0.21a <sub>x</sub>
"	" x 7/8	174	50.4	"	3.89	5.72	2.72	2.45	1 + 0.47a <sub>v</sub>	1 + 0.21a <sub>x</sub>
"	" x 3/4	162	46.9	"	3.87	5.62	2.73	2.44	1 + 0.47a <sub>v</sub>	1 + 0.22a <sub>x</sub>
"	" x 5/8	150	43.4	"	3.86	5.53	2.74	2.44	1 + 0.47a <sub>v</sub>	1 + 0.22a <sub>x</sub>
"	" x 1/2	140	39.9	"	3.84	5.43	2.75	2.43	1 + 0.48a <sub>v</sub>	1 + 0.22a <sub>x</sub>
"	" x 3/8	126	36.4	"	3.83	5.32	2.77	2.43	1 + 0.48a <sub>v</sub>	1 + 0.23a <sub>x</sub>
12 x 5	12 x 1	148	42.8	6	3.33	5.33	2.71	2.44	1 + 0.54a <sub>v</sub>	1 + 0.21a <sub>x</sub>
"	" x 7/8	138	39.8	"	3.32	5.74	2.72	2.44	1 + 0.54a <sub>v</sub>	1 + 0.21a <sub>x</sub>
"	" x 3/4	127 1/2	36.8	"	3.31	5.61	2.73	2.43	1 + 0.55a <sub>v</sub>	1 + 0.21a <sub>x</sub>
"	" x 1/2	117 1/2	33.8	"	3.30	5.51	2.75	2.43	1 + 0.55a <sub>v</sub>	1 + 0.22a <sub>x</sub>
"	" x 3/8	107 1/2	30.8	"	3.28	5.43	2.76	2.43	1 + 0.56a <sub>v</sub>	1 + 0.22a <sub>x</sub>
"	" x 1/4	97	27.8	"	3.26	5.31	2.79	2.44	1 + 0.56a <sub>v</sub>	1 + 0.23a <sub>x</sub>
10 x 6	14 x 1 1/2	229 1/2	66.7	7	3.94	5.22	2.67	2.55	1 + 0.45a <sub>v</sub>	1 + 0.24a <sub>x</sub>
"	" x 1 1/2	205 1/2	59.7	"	3.92	5.07	2.68	2.52	1 + 0.46a <sub>v</sub>	1 + 0.25a <sub>x</sub>
"	" x 1	181 1/2	52.7	"	3.91	4.91	2.70	2.49	1 + 0.46a <sub>v</sub>	1 + 0.25a <sub>x</sub>
"	" x 7/8	170	49.2	"	3.90	4.83	2.71	2.48	1 + 0.46a <sub>v</sub>	1 + 0.25a <sub>x</sub>
"	" x 3/4	158	45.7	"	3.89	4.75	2.71	2.47	1 + 0.46a <sub>v</sub>	1 + 0.26a <sub>x</sub>
"	" x 5/8	146	42.2	"	3.87	4.66	2.73	2.46	1 + 0.47a <sub>v</sub>	1 + 0.26a <sub>x</sub>
"	" x 1/2	134	38.7	"	3.86	4.57	2.74	2.45	1 + 0.47a <sub>v</sub>	1 + 0.27a <sub>x</sub>
"	" x 3/8	122	35.2	"	3.84	4.47	2.76	2.44	1 + 0.48a <sub>v</sub>	1 + 0.27a <sub>x</sub>

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2 1/2 per cent. over this must be allowed. See page 7.

Each weight per foot is for the riveted shaft only. Weight of base, &c., to be added.

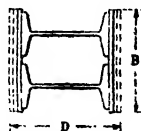
Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

We=actual eccentric load; K=relative eccentricity coefficient; Wc=equivalent concentric value; Wc=W x K.

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for a<sub>v</sub> and a<sub>x</sub> respectively.

For full explanations of tables, see notes commencing page 192.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND STANCHIONS.

Safe Concentric Loads, in Tons.  
Ends Flat

Reference Mark.	Size, D x B inches	HEIGHTS IN FEET.															
		8	12	16	20	22	24	26	28	30	32	34	36	38	40		
64 M	12 x 12	275	270	262	253	248	242	236	229	220	193	171	153	137	124		
63 M	11½ x 11	255	250	243	233	230	224	219	212	203	178	158	141	127	114		
62 M	11 x 11	235	231	224	216	212	207	201	196	186	164	145	129	116	105		
61 M	11½ x 10	215	211	205	198	191	189	181	173	169	149	132	118	105	95		
60 M	11 x 10	195	192	186	180	179	171	167	162	152	134	118	106	94	85		
59 M	10½ x 10	176	172	167	161	158	153	149	145	135	119	105	93	84	76		
50 M	10 x 10	146	142	137	129	125	121	108	91	79	76	62	55				
49 M	9½ x 9	130	126	121	115	111	107	93	80	70	61	51	44				
38 M	11 x 14	415	409	401	391	386	379	372	365	357	349	340	319	287	259		
36 M	10½ x 11	368	363	356	347	342	337	330	324	317	309	302	282	253	228		
34 M	10 x 11	322	317	311	303	299	294	283	283	277	270	263	244	219	198		
33 M	9½ x 11	299	294	289	281	277	273	268	262	256	250	244	226	202	183		
32 M	9½ x 10	275	271	266	259	255	251	246	241	236	230	224	205	184	166		
31 M	9½ x 9	252	248	243	237	233	229	224	219	214	209	201	179	161	145		
30 M	9 x 10	229	225	220	214	211	207	202	198	193	183	174	155	139	126		
29 M	8½ x 10	205	202	198	192	188	181	180	176	172	167	149	133	119	108		
22 M	9½ x 12	227	223	217	210	205	200	195	190	182	160	141	126	113	102		
21 M	9½ x 11	208	204	198	191	187	183	178	173	165	145	128	114	103	92		
20 M	9 x 11	188	181	179	173	169	165	161	156	148	130	115	103	92	83		
19 M	8½ x 11	168	165	160	154	151	147	143	139	131	115	103	90	81	73		
10 M	9 x 10	135	131	126	119	115	111	98	84	74	65	55	45				
9 M	8½ x 10	118	115	110	105	101	97	85	73	64	56	46	37				

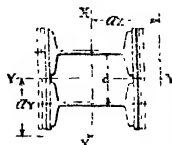
Bolts ½-in. diam. at 6-in. pitch.

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160.  
Safe loads are calculated by the Moncrieff Formula for stanchions of mild steel having "both ends flat."  
Safe loads for the condition of "both ends fixed" are identical with tabular loads on the heights to left of zigzag line.  
For other conditions and formulae, see notes commencing page 192.  
Safe loads printed in italics are for heights greater than 40 ft.  
For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND STANCHIONS.

### Composition and Properties.



Composed of		Weight per foot in lbs.	Area in square inches.	d, Centres of Webs in inches.	Radii of Gyration.		Eccentricity Coefficients.			
Two Steel Joists.	Plates, each flange to form				Axis Y-Y	Axis X-X	Web.	Flange.	Axis Y-Y	Axis X-X
10 x 5	12 x 1	144	41.6	6	3.34	4.95	2.71	2.47	1 + 0.54ax	1 + 0.25ax
"	" x 1	134	38.6	"	3.33	4.86	2.72	2.46	1 + 0.54ax	1 + 0.25ax
"	" x 1	123 1/2	35.6	"	3.32	4.77	2.73	2.45	1 + 0.54ax	1 + 0.25ax
"	" x 1	113 1/2	32.6	"	3.31	4.68	2.74	2.45	1 + 0.55ax	1 + 0.26ax
"	" x 1	103 1/2	29.6	"	3.29	4.58	2.76	2.44	1 + 0.55ax	1 + 0.28ax
"	" x 1	93	26.6	"	3.23	4.47	2.78	2.44	1 + 0.56ax	1 + 0.27ax
9 x 4	10 x 1	78 1/2	22.3	5	2.75	4.16	3.10	2.14	1 + 0.66ax	1 + 0.29ax
"	" x 1	70	19.8	"	2.73	4.06	3.13	2.15	1 + 0.67ax	1 + 0.30ax
8 x 6	14 x 1 1/2	215 1/2	62.6	7	3.94	4.33	2.19	2.61	1 + 0.45ax	1 + 0.29ax
"	" x 1 1/2	191 1/2	55.6	"	3.93	4.18	2.20	2.57	1 + 0.45ax	1 + 0.30ax
"	" x 1	167 1/2	48.6	"	3.91	4.03	2.70	2.54	1 + 0.46ax	1 + 0.31ax
"	" x 1	156	45.1	"	3.90	3.95	2.71	2.52	1 + 0.46ax	1 + 0.31ax
"	" x 1	144	41.6	"	3.89	3.87	2.72	2.51	1 + 0.46ax	1 + 0.32ax
"	" x 1	132	38.1	"	3.88	3.79	2.73	2.49	1 + 0.47ax	1 + 0.32ax
"	" x 1	120	34.6	"	3.86	3.70	2.74	2.48	1 + 0.47ax	1 + 0.33ax
"	" x 1	108	31.1	"	3.84	3.61	2.76	2.47	1 + 0.48ax	1 + 0.34ax
8 x 5	12 x 1 1/2	119 1/2	34.5	6	3.34	3.90	3.00	2.49	1 + 0.54ax	1 + 0.31ax
"	" x 1 1/2	109 1/2	31.5	"	3.33	3.81	3.02	2.47	1 + 0.54ax	1 + 0.32ax
"	" x 1	99 1/2	28.5	"	3.31	3.73	2.74	2.46	1 + 0.55ax	1 + 0.33ax
"	" x 1	89	25.5	"	3.29	3.63	2.76	2.45	1 + 0.55ax	1 + 0.33ax
8 x 4	10 x 1 1/2	72 1/2	29.6	5	2.76	3.76	2.74	2.43	1 + 0.66ax	1 + 0.32ax
"	" x 1 1/2	64	18.1	"	2.74	3.66	2.76	2.43	1 + 0.67ax	1 + 0.33ax

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 3 per cent. over this must be allowed. See page 7.

Each weight per foot is for the riveted shaft only. Weight of base, &c., to be added.

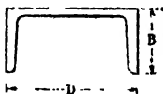
Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

W<sub>a</sub> = actual eccentric load; K = relative eccentricity coefficient; W<sub>r</sub> = equivalent concentric value. W<sub>a</sub> = W<sub>r</sub> x K.

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for  $a_v$  and  $a_x$  respectively.

For full explanations of tables, see notes commencing page 192.

REDPATH, BROWN & CO., LIMITED.



STANCHIONS.

Steel Channels.

Safe Concentric Loads, in Tons.  
Ends Flat.

Reference Mark.	Size, D x B inches.	HEIGHTS IN FEET.													
		2	3	4	5	6	7	8	9	10	11	12	13	14	
*27 N	15 x 4	81.9	81.0	79.7	78.1	76.1	73.8	71.2	68.2	61.8	51.1	42.9	36.0	31.5	
26 N	12 x 4	71.2	70.5	69.5	68.2	66.6	64.7	62.6	60.2	57.6	48.0	40.3	34.3	29.6	
*25 N	12 x 3½	61.0	63.1	61.9	60.2	58.3	56.0	53.3	46.8	37.9	31.3	26.3			
*24 N	12 x 3½	50.8	50.2	49.2	48.1	46.6	44.8	42.9	39.7	32.2	26.6	22.3	19.0		
23 N	11 x 4	61.9	64.3	63.4	62.2	60.8	59.2	57.2	55.1	52.8	45.0	37.8	32.2	27.8	
22 N	11 x 3½	58.1	57.3	56.2	54.8	53.1	51.1	48.8	44.2	35.8	29.6	24.9	21.2		
21 N	10 x 4	58.9	58.4	57.6	56.6	55.4	53.9	52.2	50.3	48.3	42.2	35.5	30.2	26.1	
*20 N	10 x 3½	55.0	54.2	53.2	51.9	50.4	48.5	46.4	43.0	34.8	28.8	24.2	20.6		
19 N	10 x 3½	45.9	45.3	44.5	43.5	42.2	40.8	39.1	37.2	30.6	25.3	21.2	18.1		
18 N	9 x 4	55.8	55.3	54.6	53.6	52.5	51.1	49.6	47.8	45.9	40.8	34.3	29.2	25.2	
*17 N	9 x 3½	49.5	48.9	48.0	46.9	45.5	43.9	42.1	40.0	32.6	26.9	22.6	19.3		
*16 N	9 x 3½	43.4	42.9	42.2	41.2	40.0	38.7	37.1	35.4	29.6	24.4	20.5	17.5		
15 N	9 x 3	37.6	36.9	35.9	34.7	33.2	31.4	28.7	21.1	17.1	14.1				
14 N	8 x 4	50.3	49.9	49.2	48.4	47.4	46.2	44.8	43.3	41.7	37.9	31.9	27.1	23.4	

The above safe loads are admitted for ratios of slenderness up to, but not exceeding 160.

Safe loads are calculated by the Moseley Formula for stanchions of mild steel having "both ends flat."

Safe loads for the conditions of "both ends fixed" are identical with tabular loads on the heights to left of zigzag line.

For other conditions and formulae, see notes commencing page 192.

Safe loads printed in italics are for heights greater than 400.

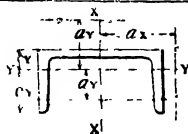
For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## STANCHIONS.

### Steel Channels.

#### Dimensions and Properties.



Size, D × B inches.	Weight per foot in lbs.	Area in square inches	Dis- tance $e_y$ inches	Radii of Gyration.		Eccentricity Coefficients.				
				Axis Y-Y	Axis X-X	Web.	Flange.	Axis Y-Y $e_y$	Axis Y-Y $e_y$	Axis X-X
15 × 4	41.94	12.331	3.065	1.08	5.53	1.74	2.84	1 + 2.60 $a_y$	1 + 0.79 $a_y$	1 + 0.25 $a_x$
12 × 4	36.47	10.727	2.969	1.13	4.51	1.84	2.77	1 + 2.34 $a_y$	1 + 0.81 $a_y$	1 + 0.30 $a_x$
12 × 3½	32.88	9.671	2.633	0.96	4.44	1.82	2.83	1 + 2.86 $a_y$	1 + 0.94 $a_y$	1 + 0.31 $a_x$
12 × 3½	26.10	7.975	2.640	0.99	4.54	1.75	2.74	1 + 2.68 $a_y$	1 + 0.87 $a_y$	1 + 0.29 $a_x$
11 × 4	33.22	9.771	2.937	1.14	4.17	1.85	2.74	1 + 2.24 $a_y$	1 + 0.81 $a_y$	1 + 0.32 $a_x$
11 × 3½	29.82	8.771	2.604	0.98	4.11	1.84	2.79	1 + 2.71 $a_y$	1 + 0.93 $a_y$	1 + 0.33 $a_x$
10 × 4	30.16	8.871	2.898	1.16	3.84	1.90	2.70	1 + 2.14 $a_y$	1 + 0.82 $a_y$	1 + 0.34 $a_x$
10 × 3½	28.21	8.296	2.567	0.99	3.77	1.88	2.76	1 + 2.60 $a_y$	1 + 0.95 $a_y$	1 + 0.35 $a_x$
10 × 3½	23.55	6.925	2.567	1.02	3.85	1.84	2.69	1 + 2.47 $a_y$	1 + 0.90 $a_y$	1 + 0.34 $a_x$
9 × 4	28.55	8.396	2.649	1.17	3.18	1.96	2.67	1 + 2.06 $a_y$	1 + 0.83 $a_y$	1 + 0.37 $a_x$
9 × 3½	25.30	7.469	2.529	1.01	3.43	1.92	2.72	1 + 2.47 $a_y$	1 + 0.95 $a_y$	1 + 0.38 $a_x$
9 × 3½	22.27	6.550	2.524	1.03	3.19	1.90	2.66	1 + 2.38 $a_y$	1 + 0.92 $a_y$	1 + 0.37 $a_x$
9 × 3	19.37	5.696	2.246	0.84	3.38	1.81	2.77	1 + 3.19 $a_y$	1 + 1.07 $a_y$	1 + 0.40 $a_x$
8 × 4	25.73	7.569	2.799	1.19	3.12	2.01	2.64	1 + 1.97 $a_y$	1 + 0.84 $a_y$	1 + 0.41 $a_x$

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

Each weight per foot is for the shaft only. Weight of base, &c., to be added.

Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

$W_e$  = actual eccentric load;  $K$  = relative eccentricity coefficient.  $W_c$  = equivalent concentric value;  $W_c = W_e \times K$ .

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for  $a_y$  and  $a_x$  respectively.

Sections marked (\*) are in our stocks.

For full explanations of tables, see notes commencing page 192.

# REDPATH, BROWN & CO., LIMITED.

## STANCHIONS.

### Steel Channels.



Safe Concentric Loads, in Tons.  
Ends Flat.

Reference Mark.	Size, D x B inches.	HEIGHTS IN FEET.											
		2	3	4	5	6	7	8	9	10	11	12	13
*13 N	8 x 3½	44.3	43.8	43.0	42.0	40.8	39.4	37.8	36.1	30.0	24.8	20.3	17.7
12 N	8 x 3	47.5	36.8	35.9	34.8	33.4	31.8	28.7	22.7	18.4	15.2		
11 N	8 x 2½	26.2	22.8	21.7	20.6	19.2	17.5	15.8	14.1	11.9			
*10 N	7 x 3½	39.5	39.0	38.3	37.5	36.5	35.2	33.9	32.4	27.6	22.8	19.2	16.3
9 N	7 x 3	31.1	33.6	32.8	31.7	30.5	29.0	26.7	21.1	17.1	14.1		
8 N	6 x 3½	31.9	34.5	34.0	33.2	32.2	31.3	30.1	28.8	25.1	20.7	17.4	14.8
*7 N	6 x 3	31.7	31.1	30.4	29.5	28.4	27.0	25.4	20.0	16.2	12.4		
*6 N	6 x 3	28.2	27.7	27.1	26.3	25.3	24.2	22.9	18.4	11.9	12.3	10.3	
5 N	6 x 2½	23.2	22.7	21.9	20.8	19.6	16.3	12.5	9.9				
*4 N	5 x 2½	21.2	20.7	20.0	19.1	18.0	15.4	11.8	9.3				
*3 N	4 x 2	15.2	14.7	13.9	12.9	9.9	7.3	5.6					
2 N	3½ x 2	12.9	12.4	11.8	10.9	8.4	6.2						
*1 N	3 x 1½	9.8	9.1	7.8	5.0								

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160. Safe loads are calculated by the Moncrieff Formulae for stanchions of mild steel having "both ends flat."

Safe loads for the condition of "both ends fixed" are identical with tabular loads on the heights to left of zigzag line.

For other conditions and formulae, see notes commencing page 132.

Safe loads printed in italics are for heights greater than 40B.

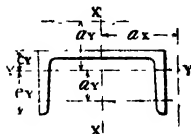
For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## STANCHIONS.

### Steel Channels.

#### Dimensions and Properties.



Size, D x B inches.	Weight per foot in lbs.	Area in square inches	Dis- tance C <sub>y</sub> inches.	Radii of Gyration.		Eccentricity Coefficients.				
				Axis Y-Y	Axis X-X	Web.	Flange	Axis Y-Y C <sub>y</sub>	Axis Y-Y C <sub>x</sub>	Axis X-X
*8 x 3½	22.72	6.682	2.489	1.03	3.09	1.97	2.68	1 + 2.36a <sub>y</sub>	1 + 0.96a <sub>y</sub>	1 + 0.42a <sub>x</sub>
8 x 3	19.30	5.675	2.156	0.87	3.07	1.94	2.70	1 + 2.83a <sub>y</sub>	1 + 1.11a <sub>y</sub>	1 + 0.43a <sub>x</sub>
8 x 2½	15.12	4.448	1.831	0.71	3.04	1.87	2.73	1 + 3.58a <sub>y</sub>	1 + 1.30a <sub>y</sub>	1 + 0.43a <sub>x</sub>
*7 x 3½	20.23	5.950	2.439	1.04	2.73	2.03	2.64	1 + 2.24a <sub>y</sub>	1 + 0.97a <sub>y</sub>	1 + 0.47a <sub>x</sub>
7 x 3	17.56	5.166	2.126	0.88	2.70	1.98	2.68	1 + 2.74a <sub>y</sub>	1 + 1.13a <sub>y</sub>	1 + 0.48a <sub>x</sub>
6 x 3½	17.90	5.266	2.381	1.06	2.37	2.12	2.60	1 + 2.12a <sub>y</sub>	1 + 1.00a <sub>y</sub>	1 + 0.53a <sub>x</sub>
*6 x 3	16.29	4.791	2.072	0.89	2.33	2.08	2.66	1 + 2.60a <sub>y</sub>	1 + 1.17a <sub>y</sub>	1 + 0.55a <sub>x</sub>
*6 x 3	14.49	4.261	2.062	0.90	2.37	2.07	2.60	1 + 2.51a <sub>y</sub>	1 + 1.14a <sub>y</sub>	1 + 0.53a <sub>x</sub>
6 x 2½	12.04	3.542	1.796	0.73	2.30	1.93	2.70	1 + 3.38a <sub>y</sub>	1 + 1.33a <sub>y</sub>	1 + 0.57a <sub>x</sub>
*5 x 2½	10.98	3.230	1.743	0.74	1.94	2.05	2.67	1 + 3.18a <sub>y</sub>	1 + 1.38a <sub>y</sub>	1 + 0.67a <sub>x</sub>
*4 x 2	7.96	2.341	1.344	0.66	1.56	2.20	2.64	1 + 3.74a <sub>y</sub>	1 + 1.83a <sub>y</sub>	1 + 0.82a <sub>x</sub>
3½ x 2	6.75	1.986	1.355	0.60	1.36	2.16	2.65	1 + 3.78a <sub>y</sub>	1 + 1.80a <sub>y</sub>	1 + 0.94a <sub>x</sub>
*3 x 1½	5.27	1.549	1.016	0.43	1.13	2.23	2.68	1 + 5.32a <sub>y</sub>	1 + 2.54a <sub>y</sub>	1 + 1.12a <sub>x</sub>

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

Each weight per foot is for the shaft only. Weight of base, &c., to be added.

Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

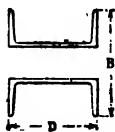
We=actual eccentric load; K=relative eccentricity coefficient; Wc=equivalent concentric value; Wc=WxK.

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for a<sub>y</sub> and a<sub>x</sub> respectively.

Sections marked (\*) are in our stocks.

For full explanations of tables, see notes commencing page 192.

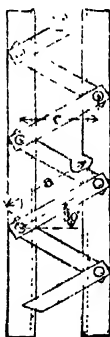
# REDPATH, BROWN & CO., LIMITED.



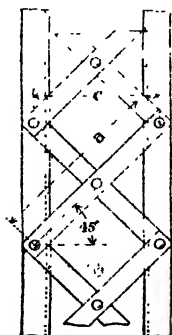
## COMPOUND STANCHIONS.

Safe Concentric Loads, in Tons.  
Ends Flat.

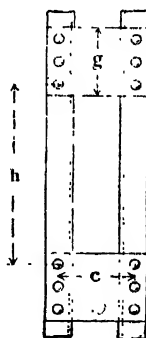
Reference Mark.	Size, D x B inches.	HEIGHTS IN FEET.													
		8	12	16	18	20	22	24	26	28	30	32	34	36	40
25 O	12 x 12	128	126	124	123	121	119	117	115	113	111	108	105	101	81.6
20 O	10 x 12	110	108	105	104	102	100	98	96	93	91	88	87	86	56.5
17 O	9 x 11	98	96	93	91	89	87	85	82	80	79	78	76	74	59.1
13 O	8 x 10	87	85	81	79	77	74	71	68	65	63	61	59	57	48.3
10 O	7 x 9	77	74	69	67	64	61	58	55	52	50	48	46	44	36.3



**SINGLE LATTICING.**  
Suitable for values of *a*, not exceeding 15 inches.



**DOUBLE LATTICING.**



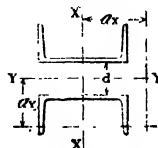
**BATTEN PLATES.**

The above safe loads are tabulated for ratio of slenderness up to, but not exceeding 160.  
Safe loads are calculated by the Moncrieff Formulae for stanchions of mild steel having "both ends flat."  
Safe loads for the condition of "both ends fixed" are identical with tabular loads on the heights to left of zigzag line.  
For other conditions and formulae, see notes commencing page 192.  
Safe loads printed in Italics are for heights greater than 40 ft.  
For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND STANCHIONS.

### Composition and Properties.



Composed of Two Steel Channels Latticed.	Weight per foot in lbs.	Area in square inches.	d. Space between Webs. Inches.	Radii of Gyration.		Eccentricity Coefficients.			
				Axis Y-Y	Axis X-X	Web.	Flange.	Axis Y-Y	Axis X-X
12 x 3½	66	19.3	6	3.98	4.44	3.15	2.83	1+0.41a <sub>v</sub>	1+0.31a <sub>x</sub>
10 x 3½	56½	16.6	5	3.57	3.77	3.22	2.76	1+0.47a <sub>v</sub>	1+0.35a <sub>x</sub>
9 x 3½	51	14.9	4	3.14	3.43	3.35	2.72	1+0.56a <sub>v</sub>	1+0.38a <sub>x</sub>
8 x 3½	45½	13.3	3	2.71	3.09	3.50	2.68	1+0.68a <sub>v</sub>	1+0.42a <sub>x</sub>
7 x 3½	40½	11.9	2	2.31	2.73	3.66	2.64	1+0.84a <sub>v</sub>	1+0.47a <sub>x</sub>

CONVENTIONAL MAXIMUM SPACING AND MINIMUM PROPORTIONS OF LATTICE BARS AND BATTEN PLATES FOR CONCENTRIC LOADING (*Am. Ry. Engineering and Maintenance of Way Assoc.*).

Depth of Channel, Inches.	12	10	9	8	7
Width of Lattice Bar, Inches.	2½	2½	2½	2½	2
Diameter of Rivet.	¾	¾	¾	¾	¾

#### SINGLE LATTICING—

Maximum angle of inclination with horizontal = 30 degrees  
 Minimum thickness = 1/40th of *a*, the diagonal centres of rivets.  
 Maximum horizontal centres of rivets, *c* = 15 inches

#### DOUBLE LATTICING—

Maximum angle of inclination with horizontal = 45 degrees  
 Minimum thickness = 1/60th of *a*, the diagonal centres of rivets.

#### BATTEN PLATES—

Maximum centres of end rivets of batten plates = *h* inches.  
 Let *l* = height of stanchion in inches, and *k* = radius of gyration of one channel  
 Then *h* =  $\frac{l}{k}$  least.  
               *k* greatest.

Minimum thickness = 1/60th of *c*, the horizontal centres of rivets  
 Minimum width *g* = *c*, the horizontal centres of rivets for end plates.  
 " " *g* = ½*c*, " " " intermediate plates.

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

Each weight per foot is for the shaft only. Weights of lattices, base, &c., to be added.

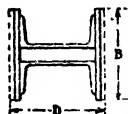
Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

W<sub>a</sub> = actual eccentric load. K = relative eccentricity coefficient; W<sub>e</sub> = equivalent concentric value; W<sub>c</sub> = W<sub>a</sub> x K.

In axial eccentricity coefficients substitute actual value of "axis of eccentricity" for *a<sub>v</sub>* and *a<sub>x</sub>* respectively.

For full explanations of tables, see notes commencing page 192.

REDPATH, BROWN & CO., LIMITED.



COMPOUND STANCHIONS.

Safe Concentric Loads, in Tons.  
Ends Flat.

Reference Mark	Size, D x B inches.	HEIGHTS IN FEET.													
		8	12	16	20	22	24	26	28	30	32	34	36	38	40
58 P	15 x 14	406	400	391	380	373	366	359	350	342	333	307	274	246	222
56 P	14½ x "	359	354	346	336	330	323	316	309	301	293	265	237	212	192
54 P	14 x "	313	308	301	292	286	280	274	268	261	252	223	199	179	161
53 P	13½ x "	290	285	278	269	264	259	253	247	240	228	202	180	162	146
52 P	13½ x "	266	262	256	247	243	237	232	226	220	205	181	162	145	131
51 P	13½ x "	243	239	233	225	221	216	211	205	199	181	160	143	128	116
50 P	13 x "	220	216	210	203	199	194	189	184	178	157	139	124	111	101
49 P	12½ x 12	185	180	173	164	158	153	133	115	100	88	78	69	7	
38 P	13 x 12	346	339	329	316	309	300	292	283	249	219	194	173	155	140
36 P	12½ x "	307	300	291	279	272	265	257	247	215	189	167	149	134	121
34 P	12 x "	267	261	253	242	236	229	222	208	181	159	141	126	113	102
33 P	11½ x "	247	242	234	224	218	212	205	188	164	144	128	114	102	92.3
32 P	11½ x "	227	222	215	205	200	194	188	169	147	129	114	102	91.7	82.7
31 P	11½ x "	207	202	195	187	181	176	170	149	130	114	101	90.4	81.1	
30 P	11 x "	188	183	176	168	163	158	150	130	113	99.4	88.0	78.5	70.5	
29 P	10½ x 10	156	150	141	130	110	92.9	79.2	68.3	59.4					

Rivets ¾-in. diam. at 6-in. pitch.

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 100.

Safe loads are calculated by the Moncrieff Formulae for stanchions of mild steel having "both ends flat."

Safe loads for the condition of "both ends fixed" are identical with tabular loads on the heights to left of zigzag line.

For other conditions and formulae, see notes commencing page 192.

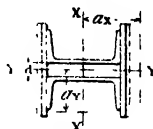
Safe loads printed in italics are for heights greater than 40D.

For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND STANCHIONS.

### Composition and Properties.



Composed of		Weight per foot in lbs.	Area in square inches.	d. Space between Webs in inches.	Radii of Gyration.		Eccentricity Coefficients.			
Two Steel Channels.	Plates, each flange to form.				Axis Y-Y	Axis X-X	Web.	Flange.	Axis Y-Y	Axis X-X
12 x 3½	14 x 1½	211	61.3	3.5	3.69	6.12	2.16	2.50	1+0.52ax	1+0.20ax
"	" x 1½	187½	54.3	"	3.64	5.94	2.19	2.49	1+0.53ax	1+0.21ax
"	" x 1	163½	47.3	"	3.58	5.75	2.23	2.48	1+0.55ax	1+0.21ax
"	" x ¾	150	43.8	"	3.54	5.64	2.26	2.48	1+0.56ax	1+0.22ax
"	" x ¾	139½	40.3	"	3.49	5.53	2.29	2.49	1+0.57ax	1+0.22ax
"	" x ¾	128	36.8	"	3.44	5.41	2.33	2.50	1+0.59ax	1+0.23ax
"	" x ¾	116	33.3	"	3.37	5.28	2.39	2.52	1+0.62ax	1+0.24ax
"	12 x ¾	99	28.3	2.5	2.74	5.06	2.40	2.59	1+0.80ax	1+0.25ax
10 x 3½	12 x 1½	181½	52.6	2.5	3.16	5.29	2.03	2.51	1+0.60ax	1+0.23ax
"	" x 1½	161	46.6	"	3.12	5.05	2.06	2.53	1+0.62ax	1+0.25ax
"	" x 1	140½	40.6	"	3.07	4.87	2.10	2.52	1+0.64ax	1+0.25ax
"	" x ¾	130½	37.6	"	3.04	4.78	2.12	2.51	1+0.65ax	1+0.26ax
"	" x ¾	120	34.6	"	3.00	4.68	2.15	2.51	1+0.67ax	1+0.26ax
"	" x ¾	110	31.6	"	2.95	4.57	2.19	2.52	1+0.69ax	1+0.27ax
"	" x ¾	99½	28.6	"	2.89	4.45	2.24	2.53	1+0.72ax	1+0.28ax
"	10 x ¾	84½	24.1	1.5	2.28	4.26	2.17	2.59	1+0.96ax	1+0.30ax

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

Each weight per foot is for the riveted shaft only. Weight of base, &c., to be added.

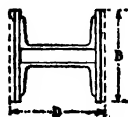
Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

W=actual eccentric load; K=relative eccentricity coefficient; We=equivalent concentric value; We=W x K.

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for  $a_y$  and  $a_x$  respectively.

For full explanations of tables, see notes commencing page 192.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND STANCHIONS.

Safe Concentric Loads, in Tons.  
Ends Flat.

Reference Mark.	Size, D x B inches.	HEIGHTS IN FEET.															
		8	12	14	16	18	20	22	24	26	28	30	32	34	36		
24 P	11 x 12	256	251	247	243	238	233	227	221	214	204	178	156	138	123		
23 P	10½ x "	236	231	228	224	219	215	209	203	197	185	161	141	125	112		
22 P	10¼ x "	216	212	208	205	201	196	191	185	180	165	144	126	112	100		
21 P	10¼ x "	197	192	189	186	182	177	173	168	162	146	127	112	99	88	5	
20 P	10 x "	177	173	170	167	163	159	155	150	145	126	110	96	85	76	4	
19 P	9½ x 10	146	140	136	132	127	122	107	90	76	66	57	6				
14 P	10 x 10	218	211	206	201	195	189	182	164	140	120	105	92	81	7		
13 P	9½ x "	201	195	191	186	180	174	168	148	126	109	95	83	74	0		
12 P	9¼ x "	185	179	175	170	165	159	153	133	113	97	85	73	74	9		
11 P	9¼ x "	168	163	159	155	150	144	139	118	100	86	75	64	66	3		
10 P	9 x "	152	147	143	139	135	130	122	102	87	73	65	56	57	6		
9 P	8½ x 9	130	123	119	115	110	105	97	79	66	56	48	9				

Rivets ½-in. diam. at 6-in. pitch.

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160.

Safe loads are calculated by the Moncrieff Formulae for stanchions of mild steel having "both ends flat."

Safe loads for the condition of "both ends fixed" are identical with tabular loads on the heights to left of zigzag line.

For other conditions and formulae, see notes commencing page 192.

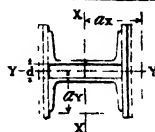
Safe loads printed in italics are for heights greater than 40D.

For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND STANCHIONS.

### Composition and Properties.



Composed of		Weight per foot in lbs.	Area in square inches.	d. Space between Webs in inches.	Radii of Gyratlon.		Eccentricity Coefficients.			
Two Steel Channels.	Plates, each flange to form.				Axis Y-Y	Axis X-X	Web.	Flange.	Axis Y-Y	Axis X-X
9 x 3½	12 x 1	135	38.9	2.5	3.11	4.47	2.06	2.52	1+0.62ax	1+0.28ax
"	" x ¾	124½	35.9	"	3.08	4.38	2.08	2.51	1+0.63ax	1+0.28ax
"	" x ¾	114½	32.9	"	3.04	4.28	2.10	2.50	1+0.65ax	1+0.29ax
"	" x ¾	104½	29.9	"	3.00	4.18	2.14	2.50	1+0.67ax	1+0.30ax
"	" x ½	94	26.9	"	2.94	4.07	2.18	2.51	1+0.70ax	1+0.30ax
"	10 x ¾	79	22.4	1.5	2.33	3.90	2.11	2.57	1+0.92ax	1+0.32ax
8 x 3½	10 x 1	116	33.3	1.5	2.58	4.00	1.88	2.56	1+0.75ax	1+0.31ax
"	" x ¾	107½	30.8	"	2.55	3.91	1.90	2.55	1+0.77ax	1+0.32ax
"	" x ¾	99	28.3	"	2.52	3.82	1.93	2.54	1+0.79ax	1+0.33ax
"	" x ¾	90½	25.8	"	2.48	3.75	1.95	2.54	1+0.81ax	1+0.33ax
"	" x ½	82	23.3	"	2.44	3.63	1.99	2.54	1+0.84ax	1+0.34ax
"	9 x ¾	71	20.1	1	2.12	3.49	1.93	2.57	1+1.01ax	1+0.36ax

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

Each weight per foot is for the riveted shaft only. Weight of base, &c., to be added.

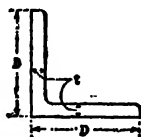
Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

We=actual eccentric load; K=relative eccentricity coefficient; Wc=equivalent concentric value; Wc=WexK.

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for ax and ax respectively.

For full explanations of tables, see notes commencing page 192.

# REDPATH, BROWN & CO., LIMITED.



## STANCHIONS (or STRUTS). Steel Equal Angles.

Safe Concentric<sup>a</sup> Loads, in Tons.  
Ends Flat.

Reference Mark.	Size, D x B x t inches.	HEIGHTS IN FEET.													
		2	3	4	5	6	7	8	9	10	11	12	13	14	
14g Q	6 x 6 x $\frac{3}{8}$	56.1	55.6	54.8	53.9	52.7	51.3	49.8	48.0	46.1	44.0	43.4	42.9	42.5	
14f Q	" x $\frac{5}{8}$	47.3	46.8	46.2	45.4	44.4	43.3	42.0	40.6	39.0	37.8	37.2	36.9	36.5	
14e Q	" x $\frac{1}{2}$	38.2	37.9	37.4	36.7	36.0	35.0	34.0	32.8	31.5	30.1	29.6	29.1	28.7	
13g Q	5 x 5 x $\frac{3}{8}$	45.9	45.3	44.4	43.2	41.8	40.1	38.3	36.5	34.7	32.9	32.5	32.1	31.7	
13f Q	" x $\frac{5}{8}$	38.8	38.3	37.6	36.6	35.5	34.1	32.6	30.9	29.5	28.1	27.6	27.1	26.7	
13e Q	" x $\frac{1}{2}$	31.5	31.0	30.4	29.7	28.7	27.7	26.4	25.0	23.9	22.9	22.4	22.0	21.6	
12g Q	4½ x 4½ x $\frac{3}{8}$	40.8	40.1	39.1	37.8	36.2	34.3	32.9	31.5	30.0	28.7	28.2	27.7	27.3	
12f Q	" x $\frac{5}{8}$	34.6	34.0	33.2	32.1	30.8	29.3	27.6	26.3	25.0	23.9	23.4	22.9	22.5	
12e Q	" x $\frac{1}{2}$	28.1	27.6	26.9	26.0	25.0	23.8	22.4	21.0	20.0	19.4	18.9	18.4	18.0	
11g Q	4 x 4 x $\frac{3}{8}$	35.7	34.9	33.7	32.3	30.5	28.5	26.3	24.5	23.0	21.7	21.2	20.7	20.3	
11f Q	" x $\frac{5}{8}$	30.3	29.6	28.7	27.5	26.1	24.3	22.8	21.5	20.3	19.1	18.6	18.1	17.7	
11e Q	" x $\frac{1}{2}$	24.7	24.1	23.4	22.4	21.2	19.3	17.8	16.8	15.7	14.8	14.3	13.8	13.4	
11d Q	" x $\frac{3}{8}$	18.8	18.4	17.8	17.1	16.2	15.1	14.0	13.1	12.2	11.5	11.0	10.5	10.1	
10f Q	3½ x 3½ x $\frac{5}{8}$	26.1	25.3	24.3	23.0	21.4	19.6	17.2	15.5	14.0	12.8	12.3	11.8	11.4	
10e Q	" x $\frac{1}{2}$	21.3	20.7	19.8	18.8	17.5	16.0	14.5	13.5	12.5	11.8	11.3	10.8	10.4	
10d Q	" x $\frac{3}{8}$	16.3	15.8	15.2	14.3	13.4	12.4	11.4	10.6	9.9	9.3	8.8	8.3	7.9	

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160.

Safe loads are calculated by the Moncrieff Formulae for stanchions of mild steel having "both ends flat."

Safe loads for the condition of "both ends fixed" are identical with tabular loads on the heights to left of zigzag line.

For other conditions and formulae, see notes commencing page 192.

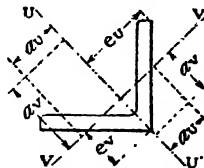
For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## STANCHIONS (or STRUTS).

### Steel Equal Angles.

#### Dimensions and Properties.



Size, D x B x t inches.	Weight per foot in lbs.	Area in square inches	Distances in inches		Radii of Gyration.		Eccentricity Coefficients.	
			$e_v$	$e_u$	Axis V-V	Axis U-U	Axis V-V	Axis U-U
6 x 6 x $\frac{3}{8}$	28.70	8.441	2.49	4.24	1.17	2.28	$1+1.82a_v$	$1+0.82a_u$
" x 6 x $\frac{5}{16}$	24.18	7.113	2.42	4.24	1.18	2.30	$1+1.74a_v$	$1+0.81a_u$
" x 6 x $\frac{1}{2}$	19.55	5.753	2.35	4.24	1.18	2.32	$1+1.69a_v$	$1+0.79a_u$
5 x 5 x $\frac{3}{8}$	23.59	6.939	2.14	3.54	0.98	1.88	$1+2.32a_v$	$1+1.00a_u$
" x 5 x $\frac{5}{16}$	19.92	5.860	2.07	3.54	0.98	1.89	$1+2.16a_v$	$1+0.98a_u$
" x 5 x $\frac{1}{2}$	16.15	4.751	2.00	3.54	0.98	1.92	$1+2.09a_v$	$1+0.96a_u$
$4\frac{1}{2}$ x $4\frac{1}{2}$ x $\frac{3}{8}$	21.05	6.189	1.96	3.18	0.85	1.69	$1+2.72a_v$	$1+1.12a_u$
" x $4\frac{1}{2}$ x $\frac{5}{16}$	17.80	5.236	1.90	3.18	0.87	1.70	$1+2.51a_v$	$1+1.10a_u$
" x $4\frac{1}{2}$ x $\frac{1}{2}$	14.46	4.252	1.83	3.18	0.87	1.72	$1+2.42a_v$	$1+1.07a_u$
4 x 4 x $\frac{3}{8}$	18.49	5.437	1.79	2.83	0.76	1.48	$1+3.10a_v$	$1+1.28a_u$
" x 4 x $\frac{5}{16}$	15.66	4.609	1.72	2.83	0.77	1.50	$1+2.91a_v$	$1+1.26a_u$
" x 4 x $\frac{1}{2}$	12.75	3.750	1.66	2.83	0.77	1.52	$1+2.80a_v$	$1+1.22a_u$
" x 4 x $\frac{3}{4}$	9.72	2.859	1.59	2.83	0.78	1.54	$1+2.61a_v$	$1+1.19a_u$
$3\frac{1}{2}$ x $3\frac{1}{2}$ x $\frac{5}{16}$	13.55	3.985	1.55	2.47	0.68	1.29	$1+3.35a_v$	$1+1.47a_u$
" x $3\frac{1}{2}$ x $\frac{3}{8}$	11.05	3.251	1.48	2.47	0.68	1.32	$1+3.21a_v$	$1+1.43a_u$
" x $3\frac{1}{2}$ x $\frac{1}{2}$	8.45	2.485	1.41	2.47	0.68	1.34	$1+3.06a_v$	$1+1.38a_u$

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

Each weight per foot is for the shaft only. Weight of connections, &c., to be added.

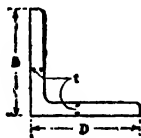
Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

$W_e$  = actual eccentric load;  $K$  = relative eccentricity coefficient;  $W_c$  = equivalent concentric value;  $W_c = W_e \times K$ .

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for  $a_v$  and  $a_u$  respectively.

For full explanations of tables, see notes commencing page 192.

REDPATH, BROWN & CO., LIMITED.



**STANCHIONS (or STRUTS).**  
**Steel Equal Angles.**

Safe Concentric \*Loads, in Tons.  
Ends Flat.

Reference Mark.	Size, D x B x t inches.	HEIGHTS IN FEET.							
		2	3	4	5	6	7	—	—
9f Q	3 x 3 x $\frac{1}{8}$	21.8	21.0	19.8	18.3	13.3	9.8	—	—
9e Q	" x $\frac{1}{2}$	17.9	17.2	16.2	15.0	10.9	8.0	—	—
9d Q	" x $\frac{3}{8}$	13.7	13.2	12.4	11.5	8.4	6.1	—	—
9c Q	" x $\frac{1}{4}$	11.5	11.1	10.4	9.6	7.0	5.2	—	—
9b Q	" x $\frac{1}{8}$	9.3	9.0	8.5	7.9	5.9	4.3	—	—
7e Q	2½ x 2½ x $\frac{1}{2}$	14.4	13.5	12.4	8.8	6.1		—	—
7d Q	" x $\frac{3}{8}$	11.1	10.4	9.5	6.8	4.7		—	—
7c Q	" x $\frac{1}{4}$	9.3	8.8	8.0	5.7	4.0		—	—
7b Q	" x $\frac{1}{8}$	7.6	7.1	6.5	4.6	3.2		—	—
6c Q	2½ x 2½ x $\frac{1}{8}$	8.3	7.7	6.4	4.1			—	—
6b Q	" x $\frac{1}{4}$	6.7	6.2	5.2	3.3			—	—
5b Q	2 x 2 x $\frac{1}{2}$	5.9	5.3	3.8	2.4			—	—
5a Q	" x $\frac{1}{8}$	4.5	4.1	2.9	1.8			—	—

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160.

Safe loads are calculated by the Moncrieff Formulae for stanchions of mild steel having "both ends flat."

Safe loads for the condition of "both ends fixed" are identical with tabular loads on the heights to left of zigzag line.

For other conditions and formulae, see notes commencing page 192.

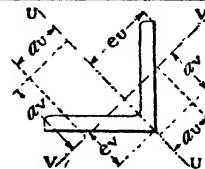
For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## STANCHIONS (or STRUTS).

### Steel Equal Angles.

#### Dimensions and Properties.



Size, D × B × t inches.	Weight per foot in lbs.	Area in square inches.	Distances in inches.		Radii of Gyration.		Eccentricity Coefficients	
			e <sub>v</sub>	e <sub>u</sub>	Axis V-V	Axis U-U	Axis V-V	Axis U-U
3 × 3 × $\frac{5}{16}$	11.43	3.362	1.37	2.12	0.58	1.09	1+4.07a <sub>v</sub>	1+1.76a <sub>u</sub>
" × $\frac{1}{2}$	9.36	2.753	1.31	2.12	0.58	1.12	1+3.89a <sub>v</sub>	1+1.70a <sub>u</sub>
" × $\frac{3}{8}$	7.18	2.112	1.24	2.12	0.58	1.13	1+3.69a <sub>v</sub>	1+1.64a <sub>u</sub>
" × $\frac{1}{4}$	6.05	1.776	1.21	2.12	0.58	1.15	1+3.60a <sub>v</sub>	1+1.61a <sub>u</sub>
" × $\frac{1}{8}$	4.90	1.440	1.17	2.12	0.59	1.15	1+3.37a <sub>v</sub>	1+1.60a <sub>u</sub>
2½ × 2½ × $\frac{1}{2}$	7.65	2.250	1.13	1.77	0.48	0.91	1+4.90a <sub>v</sub>	1+2.12a <sub>u</sub>
" × $\frac{3}{8}$	5.89	1.734	1.06	1.77	0.48	0.93	1+4.62a <sub>v</sub>	1+2.02a <sub>u</sub>
" × $\frac{1}{4}$	4.96	1.460	1.03	1.77	0.48	0.94	1+4.49a <sub>v</sub>	1+1.97a <sub>u</sub>
" × $\frac{1}{8}$	4.04	1.187	0.99	1.77	0.48	0.95	1+4.32a <sub>v</sub>	1+1.94a <sub>u</sub>
2¼ × 2¼ × $\frac{1}{4}$	4.45	1.310	0.94	1.59	0.43	0.84	1+5.11a <sub>v</sub>	1+2.22a <sub>u</sub>
" × $\frac{1}{8}$	3.61	1.061	0.91	1.59	0.44	0.85	1+4.72a <sub>v</sub>	1+2.20a <sub>u</sub>
2 × 2 × $\frac{1}{4}$	3.19	0.940	0.82	1.41	0.39	0.74	1+5.39a <sub>v</sub>	1+2.56a <sub>u</sub>
" × $\frac{1}{8}$	2.43	0.720	0.78	1.41	0.39	0.75	1+5.12a <sub>v</sub>	1+2.49a <sub>u</sub>

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

Each weight per foot is for the shaft only. Weight of connections, &c., to be added.

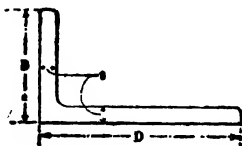
Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

We=actual eccentric load; K=relative eccentricity coefficient; We=equivalent concentric value; Wc=Wc×K.

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for a<sub>v</sub> and a<sub>u</sub> respectively.

For full explanations of tables, see notes commencing page 192.

# REDPATH, BROWN & CO., LIMITED.



## STANCHIONS (or STRUTS). Steel Unequal Angles.

Safe Concentric Loads, in Tons.  
Ends Flat.

Reference Mark.	Size, D x B x t inches.	HEIGHTS IN FEET.									
		2	3	4	5	6	7	8	9	10	11
25g R	$7 \times 3\frac{1}{2} \times \frac{3}{4}$	48.0	46.8	45.2	43.1	40.6	33.8	25.9	20.4		
25f R	" $\times \frac{5}{8}$	40.5	39.6	38.2	36.5	34.4	29.3	22.4	17.7		
25e R	" $\times \frac{1}{2}$	32.8	32.1	31.0	29.6	27.9	23.7	18.2	14.4		
21f R	$6 \times 4 \times \frac{5}{8}$	38.7	38.0	37.1	35.8	34.3	32.6	28.8	22.7	18.4	15.2
21e R	" $\times \frac{1}{2}$	31.4	30.8	30.0	29.0	27.8	26.4	23.3	18.4	14.9	12.3
20f R	$6 \times 3\frac{1}{2} \times \frac{5}{8}$	36.5	35.6	34.4	32.9	31.1	27.1	20.7	16.4	13.3	
20e R	" $\times \frac{1}{2}$	29.6	28.9	27.9	26.7	25.2	22.0	16.8	13.3	10.7	
20d R	" $\times \frac{3}{8}$	22.5	22.0	21.3	20.4	19.3	17.2	13.1	10.4	8.4	
63f R	$6 \times 3 \times \frac{5}{8}$	34.1	33.0	31.4	29.4	24.5	18.0	13.8			
63e R	" $\times \frac{1}{2}$	27.7	26.8	25.5	23.9	19.9	14.6	11.2			
63d R	" $\times \frac{3}{8}$	21.1	20.4	19.5	18.3	15.6	11.5	8.8			

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160.

Safe loads are calculated by the Moncrieff Formulae for stanchions of mild steel having "both ends flat."

Safe loads for the condition of "both ends fixed" are identical with tabular loads on the heights to left of zigzag line.

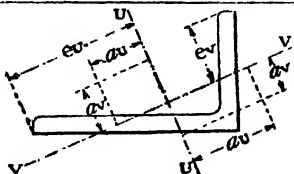
For other conditions and formulae, see notes commencing page 192.

For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## STANCHIONS (or STRUTS). Steel Unequal Angles.

Dimensions and Properties.



Size, D × B × t inches.	Weight per foot in lbs.	Area in square inches.	Distances in inches		Radii of Gyration.		Eccentricity Coefficients.	
			$e_v$	$e_u$	Axis V-V	Axis U-U	Axis V-V	Axis U-U
$7 \times 3\frac{1}{2} \times \frac{3}{8}$	24.86	7.313	2.03	4.48	0.73	2.27	$1+3.81a_v$	$1+0.87a_u$
" $\times \frac{5}{8}$	20.98	6.172	2.05	4.51	0.74	2.29	$1+3.75a_v$	$1+0.86a_u$
" $\times \frac{1}{2}$	17.00	5.000	2.07	4.55	0.74	2.31	$1+3.78a_v$	$1+0.86a_u$
$6 \times 4 \times \frac{5}{8}$	19.92	5.860	2.08	4.06	0.86	2.01	$1+2.82a_v$	$1+1.00a_u$
" $\times \frac{1}{2}$	16.15	4.750	2.08	4.09	0.86	2.03	$1+2.82a_v$	$1+0.99a_u$
$6 \times 3\frac{1}{2} \times \frac{5}{8}$	18.87	5.550	1.94	3.96	0.74	1.98	$1+3.49a_v$	$1+1.01a_u$
" $\times \frac{1}{2}$	15.31	4.502	1.92	3.99	0.75	2.00	$1+3.42a_v$	$1+1.00a_u$
" $\times \frac{3}{8}$	11.64	3.424	1.96	4.02	0.76	2.01	$1+3.39a_v$	$1+1.00a_u$
$6 \times 3 \times \frac{5}{8}$	17.80	5.236	1.76	3.84	0.63	1.94	$1+4.43a_v$	$1+1.02a_u$
" $\times \frac{1}{2}$	14.46	4.252	1.76	3.88	0.63	1.96	$1+4.46a_v$	$1+1.01a_u$
" $\times \frac{3}{8}$	11.00	3.236	1.80	3.91	0.64	1.98	$1+4.38a_v$	$1+1.00a_u$

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

Each weight per foot is for the shaft only. Weight of connections, &c., to be added.

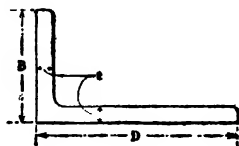
Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

$W_e$ =actual eccentric load;  $K$ =relative eccentricity coefficient;  $W_c$ =equivalent concentric value;  $W_c = W_e \times K$ .

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for  $a_v$  and  $a_u$  respectively.

For full explanations of tables, see notes commencing page 182.

# REDPATH, BROWN & CO., LIMITED.



## STANCHIONS (or STRUTS). Steel Unequal Angles.

Safe Concentric Loads, in Tons.  
Ends Flat.

Reference Mark.	Size, D x B x t inches.	HEIGHTS IN FEET.									
		2	3	4	5	6	7	8	9	10	11
17f R	5 x 4 x $\frac{5}{8}$	34.5	33.9	33.0	31.8	30.4	28.7	23.9	18.9	15.3	12.7
17e R	" x $\frac{1}{2}$	28.0	27.5	26.8	25.9	24.7	23.4	19.9	15.7	12.7	10.5
17d R	" x $\frac{3}{8}$	21.3	21.0	20.4	19.7	18.9	17.9	15.5	12.3	9.9	8.2
15f R	5 x 3 x $\frac{5}{8}$	30.1	29.1	27.8	26.1	22.3	16.4	12.5			
15e R	" x $\frac{1}{2}$	24.5	23.7	22.6	21.2	18.1	13.3	10.2			
15d R	" x $\frac{3}{8}$	18.7	18.1	17.3	16.2	14.3	10.5	8.0			
11e R	4 x 3 x $\frac{1}{2}$	21.2	20.5	19.5	18.3	15.2	11.2	8.6			
11d R	" x $\frac{3}{8}$	16.2	15.7	15.0	14.0	12.0	8.8	6.7			
7d R	3 x 2 $\frac{1}{2}$ x $\frac{3}{8}$	12.4	11.7	10.9	8.8	6.1					
7e R	" x $\frac{1}{4}$	10.4	9.9	9.2	7.4	5.2					

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 180. Safe loads are calculated by the Moncrieff Formulae for stanchions of mild steel having "both ends flat."

Safe loads for the condition of "both ends fixed" are identical with tabular loads on the heights to left of zigzag line.

For other conditions and formulae, see notes commencing page 192.

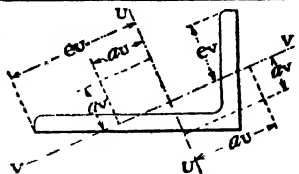
For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## STANCHIONS (or STRUTS).

### Steel Unequal Angles.

#### Dimensions and Properties.



Size, D x B x t inches.	Weight per foot in lbs.	Area in square inches.	Distances in inches.		Radii of Gyration.		Eccentricity Coefficients.	
			$e_v$	$e_u$	Axis V-V	Axis U-U	Axis V-V	Axis U-U
5 x 4 x $\frac{5}{16}$	17.80	5.236	1.81	3.46	0.83	1.74	$1 + 2.63a_v$	$1 + 1.15a_u$
" x $\frac{1}{2}$	14.46	4.252	1.83	3.48	0.84	1.75	$1 + 2.60a_v$	$1 + 1.13a_u$
" x $\frac{3}{8}$	11.00	3.236	1.82	3.49	0.85	1.77	$1 + 2.52a_v$	$1 + 1.12a_u$
5 x 3 x $\frac{5}{16}$	15.67	4.609	1.65	3.30	0.64	1.64	$1 + 4.05a_v$	$1 + 1.23a_u$
" x $\frac{1}{2}$	12.75	3.749	1.65	3.32	0.64	1.66	$1 + 4.02a_v$	$1 + 1.21a_u$
" x $\frac{3}{8}$	9.72	2.859	1.67	3.36	0.65	1.67	$1 + 3.95a_v$	$1 + 1.20a_u$
4 x 3 x $\frac{1}{2}$	11.05	3.251	1.45	2.75	0.63	1.36	$1 + 3.66a_v$	$1 + 1.48a_u$
" x $\frac{3}{8}$	8.45	2.485	1.45	2.77	0.64	1.38	$1 + 3.54a_v$	$1 + 1.45a_u$
3 x 2 $\frac{1}{2}$ x $\frac{5}{16}$	6.53	1.921	1.11	2.09	0.52	1.05	$1 + 4.10a_v$	$1 + 1.90a_u$
" x $\frac{3}{8}$	5.51	1.620	1.10	2.10	0.52	1.06	$1 + 4.05a_v$	$1 + 1.87a_u$

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

Each weight per foot is for the shaft only. Weight of connections, &c., to be added.

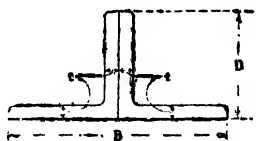
Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

We=actual eccentric load; K=relative eccentricity coefficient; We=equivalent concentric value;  $W_c = W \times K$ .

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for  $a_v$  and  $a_u$  respectively.

For full explanations of tables, see notes commencing page 192.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND STANCHIONS (or STRUTS).

Two Steel Equal Angles Back to Back.

Safe Concentric Loads, in Tons.  
Ends Flat.

Reference Mark.	Size, D x B inches.	HEIGHTS IN FEET.															
		2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	
14g S	6 x 12	113	112	111	111	110	109	107	106	104	102	100	98	295	993	490	9
14f S	"	95	094	094	103	492	691	790	089	488	086	484	883	081	079	076	8
14e S	"	76	876	576	175	674	974	273	372	571	270	068	767	265	764	162	3
13g S	5 x 10	82	591	991	290	289	187	786	184	382	380	177	775	267	358	751	6
13f S	"	78	177	777	176	375	374	172	871	369	767	865	963	858	050	514	4
13e S	"	63	363	062	561	961	160	259	157	956	655	153	651	947	841	636	6
12g S	4 1/2 x 9	82	481	881	079	978	677	175	373	371	168	665	455	748	141	936	8
12f S	"	69	769	268	567	766	665	363	862	160	358	356	248	141	536	137	7
12e S	"	56	656	255	755	054	153	151	950	649	147	545	839	834	429	926	3
11g S	4 x 8	72	371	670	769	568	066	264	262	059	653	244	738	132	828	0	
11f S	"	61	360	760	058	957	756	354	652	850	746	238	833	128	524	8	
11e S	"	49	949	448	848	047	045	944	643	141	438	332	227	423	620	618	1
11d S	"	38	037	737	236	635	935	134	133	031	829	925	121	418	516	114	1
10f S	3 1/2 x 7	52	852	251	350	248	847	145	243	236	330	025	221	5			
10e S	"	43	142	641	941	039	938	637	135	430	425	121	118	0			
10d S	"	33	032	632	131	430	529	628	527	223	819	716	514	112	1		

Rivets 3/4-in. diam. at 6-in. pitch.

Rivets 3/4-in. diam. at 6-in. pitch.

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160. Safe loads are calculated by the Moncrieff Formulae for stanchions of mild steel having "both ends flat."

Safe loads for the condition of "both ends fixed" are identical with tabular loads on the heights to left of zigzag line.

For other conditions and formulæ, see notes commencing page 192

Safe loads printed in italics are for heights greater than 40D.

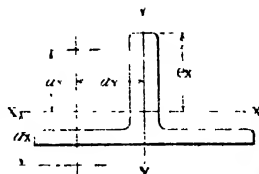
For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND STANCHIONS (or STRUTS).

Two Steel Equal Angles Back to Back.

Dimensions and Properties.



Composed of Two Equal Angles.	Weight per foot in lbs.	Area in square inches.	Distance $C_x$ inches.	Radii of Gyration.		Eccentricity Coefficients.	
				Axis Y-Y	Axis X-X	Axis Y-Y	Axis X-X
6 / 6 x 3/8	59	16.88	4.24	2.53	1.81	1 + 0.94a <sub>v</sub>	1 + 1.29a <sub>x</sub>
" x 3/8	49 1/2	14.22	4.29	2.50	1.83	1 + 0.96a <sub>v</sub>	1 + 1.28a <sub>x</sub>
" x 1/2	40 1/2	11.50	4.34	2.48	1.84	1 + 0.98a <sub>v</sub>	1 + 1.28a <sub>x</sub>
5 x 5 x 3/8	48 1/2	13.87	3.49	2.12	1.49	1 + 1.11a <sub>v</sub>	1 + 1.56a <sub>x</sub>
" x 3/8	41	11.72	3.54	2.19	1.51	1 + 1.13a <sub>v</sub>	1 + 1.55a <sub>x</sub>
" x 1/2	33 1/2	9.50	3.58	2.08	1.52	1 + 1.16a <sub>v</sub>	1 + 1.55a <sub>x</sub>
4 1/2 x 4 1/2 x 3/8	43	12.38	3.11	1.92	1.34	1 + 1.21a <sub>v</sub>	1 + 1.74a <sub>x</sub>
" x 3/8	36 1/4	10.47	3.16	1.90	1.35	1 + 1.24a <sub>v</sub>	1 + 1.73a <sub>x</sub>
" x 1/2	29 1/2	8.50	3.21	1.88	1.36	1 + 1.28a <sub>v</sub>	1 + 1.72a <sub>x</sub>
4 x 4 x 3/8	38	10.87	2.74	1.73	1.18	1 + 1.34a <sub>v</sub>	1 + 1.97a <sub>x</sub>
" x 3/8	32	9.22	2.78	1.70	1.19	1 + 1.38a <sub>v</sub>	1 + 1.96a <sub>x</sub>
" x 1/2	26 1/2	7.50	2.83	1.68	1.20	1 + 1.42a <sub>v</sub>	1 + 1.95a <sub>x</sub>
" x 3/4	20	5.72	2.88	1.66	1.22	1 + 1.46a <sub>v</sub>	1 + 1.93a <sub>x</sub>
3 1/2 x 3 1/2 x 3/8	28	7.97	2.41	1.50	1.03	1 + 1.55a <sub>v</sub>	1 + 2.25a <sub>x</sub>
" x 3/8	23	6.50	2.45	1.48	1.05	1 + 1.60a <sub>v</sub>	1 + 2.23a <sub>x</sub>
" x 1/2	17 1/2	4.97	2.50	1.45	1.06	1 + 1.65a <sub>v</sub>	1 + 2.22a <sub>x</sub>

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2 1/2 per cent. over this must be allowed. See page 7.

Each weight per foot is for the riveted shaft only. Weight of connections, &c., to be added.

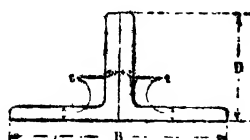
Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

We=actual eccentric load; K=relative eccentricity coefficient; We=equivalent concentric value; We=We x K.

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for a<sub>v</sub> and a<sub>x</sub> respectively.

For full explanations of tables, see notes commencing page 132

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND STANCHIONS (or STRUTS).

Two Steel Equal Angles Back to Back.

Safe Concentric Loads, in Tons.  
Ends Flat.

Reference Mark	Size, D x B inches	HEIGHTS IN FEET.										
		2	3	4	5	6	7	8	9	10	11	12
9f S	3 0	44.4	43.7	42.6	41.5	40.6	37.7	34.3	27.1	22.0	18.2	
9c S	"	36.1	35.8	35.0	33.9	32.6	31.1	29.0	22.9	18.5	15.1	
9d S	"	27.9	27.5	26.9	26.0	25.1	23.9	22.7	18.1	14.6	12.1	10.1
9e S	"	23.5	23.1	22.6	21.9	21.1	20.2	19.1	15.4	12.5	10.5	8.6
9b S	"	19.0	18.7	18.3	17.8	17.1	16.4	15.5	12.6	10.2	8.5	7.1
7e S	2½ x 5	29.5	28.8	27.8	26.5	25.0	20.8	16.0	12.6			
7d S	"	22.8	22.2	21.5	20.5	19.4	16.6	12.7	10.0			
7c S	"	19.2	18.7	18.1	17.3	16.4	14.2	10.9	8.6	7.0		
7b S	"	15.6	15.2	14.7	14.1	13.3	11.7	9.0	7.1	5.7		
6c S	2½ x 4½	14.8	14.3	13.7	13.0	12.0	8.8	6.7				
6b S	"	13.9	13.5	12.9	12.2	11.4	8.5	6.5	5.1			
5b S	2 x 4	12.2	11.8	11.1	10.3	7.9	5.8					
5a S	"	9.4	9.0	8.5	7.9	6.1	4.5	3.4				

Rivets ½-in. diam. at 6 in. pitch.

The above safe loads are tabulated for ratios of thinness up to, but not exceeding 160.  
Safe loads are calculated by the Moncrieff Formula for stanchions of mild steel having "both ends flat."

Safe loads for the conditions of "both ends fixed" are identical with tabular loads on the heights to left of zigzag line.

For other conditions and formulae, see notes commencing page 192.

Safe loads printed in italics are for heights greater than 40D.

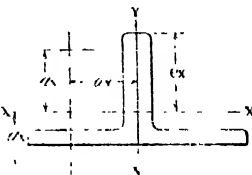
For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND STANCHIONS (or STRUTS).

Two Steel Equal Angles Back to Back.

Dimensions and Properties.



Composed of Two Equal Angles.	Weight per foot in lbs.	Area in square inches	Distance $e_x$ inches.	Radii of Gyration.		Eccentricity Coefficients.	
				Axis Y-Y	Axis X-X	Axis Y-Y	Axis X-X
3 x 3 x $\frac{5}{8}$	23 $\frac{1}{2}$	6.72	2.03	1.30	0.87	1 + 1.76 $a_v$	1 + 2.64 $a_x$
" x $\frac{3}{4}$	19 $\frac{1}{2}$	5.50	2.08	1.28	0.89	1 + 1.82 $a_v$	1 + 2.62 $a_x$
" x $\frac{7}{8}$	15	4.22	2.12	1.26	0.90	1 + 1.89 $a_v$	1 + 2.60 $a_x$
" x $\frac{7}{8}$	13	3.55	2.15	1.24	0.91	1 + 1.93 $a_v$	1 + 2.60 $a_x$
" x $\frac{1}{2}$	10 $\frac{1}{2}$	2.88	2.17	1.23	0.91	1 + 1.97 $a_v$	1 + 2.60 $a_x$
2 $\frac{1}{2}$ x 2 $\frac{1}{2}$ x $\frac{1}{2}$	16	4.50	1.70	1.08	0.73	1 + 2.14 $a_v$	1 + 3.18 $a_x$
" x $\frac{3}{4}$	12 $\frac{1}{2}$	3.47	1.75	1.06	0.74	1 + 2.24 $a_v$	1 + 3.16 $a_x$
" x $\frac{7}{8}$	10 $\frac{1}{2}$	2.92	1.77	1.04	0.75	1 + 2.29 $a_v$	1 + 3.15 $a_x$
" x $\frac{1}{2}$	8 $\frac{1}{2}$	2.37	1.80	1.03	0.75	1 + 2.35 $a_v$	1 + 3.15 $a_x$
2 $\frac{1}{2}$ x 2 $\frac{1}{2}$ x $\frac{3}{4}$	9 $\frac{1}{2}$	2.26	1.58	0.94	0.67	1 + 2.51 $a_v$	1 + 3.51 $a_x$
" x $\frac{1}{2}$	7 $\frac{1}{2}$	2.12	1.61	0.93	0.68	1 + 2.58 $a_v$	1 + 3.49 $a_x$
2 x 2 x $\frac{1}{2}$	7	1.88	1.42	0.83	0.59	1 + 2.88 $a_v$	1 + 3.97 $a_x$
" x $\frac{3}{4}$	5 $\frac{1}{2}$	1.44	1.45	0.81	0.60	1 + 3.00 $a_v$	1 + 4.01 $a_x$

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2 $\frac{1}{2}$  per cent. over this must be allowed. See page 7.

Each weight per foot is for the riveted shaft only. Weight of connections, &c., to be added.

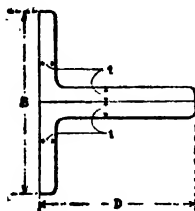
Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

We=actual eccentric load; K=relative eccentricity coefficient; Wc=equivalent concentric value; Wc=We x K.

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for  $a_v$  and  $a_x$  respectively.

For full explanations of tables, see notes commencing page 192.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND STANCHIONS (or STRUTS).

Two Steel Unequal Angles Back to Back.

Short Legs Outstanding.

Safe Concentric Loads, in Tons.  
Ends Flat.

Reference Mark.	Size, D × B inches.	HEIGHTS IN FEET.															
		2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	
25g T	7 × 7	97.3	96.5	95.5	94.3	93.2	92.1	90.0	87.6	84.9	81.9	78.7	67.1	57.2	49.3	43.0	37.8
25f T	"	82.1	81.4	80.4	79.1	77.6	75.7	73.6	71.2	68.6	64.7	54.3	46.3	39.9	34.8	30.5	
25e T	"	66.5	65.9	65.0	64.0	62.7	61.1	59.3	57.3	55.1	50.4	42.4	36.1	31.1	27.1		
21f T	6 × 8	78.1	77.7	77.1	76.3	75.3	74.2	72.9	71.4	69.7	67.9	66.0	63.9	58.4	50.9	44.7	
21e T	"	63.3	63.0	62.4	61.8	61.0	60.0	59.0	57.7	56.3	54.8	53.2	51.4	46.0	40.1	35.2	
20f T	6 × 7	73.8	73.3	72.5	71.4	70.2	68.7	66.9	65.0	62.9	60.5	54.2	46.2	39.8	34.7	30.5	
20e T	"	59.9	59.4	58.7	57.7	56.6	55.6	54.4	53.2	51.9	48.8	42.5	36.2	31.2	27.2	23.9	
20d T	"	45.5	45.2	44.6	43.9	43.1	42.1	41.0	39.7	38.3	36.7	31.1	26.5	22.8	18.8	17.5	
63/ T	6 × 6	69.5	68.7	67.6	66.1	64.4	62.3	60.0	57.4	55.0	53.4	6.34	9.29	8.25	6		
63e T	"	56.4	55.7	54.8	53.3	52.1	50.3	48.3	46.1	43.9	42.2	27.1	23.1				
63d T	"	42.9	42.4	41.6	40.6	39.4	38.1	36.5	34.7	32.8	32.3	4.19	7.16	8			
Rivets 3-in. diam. at 6-in. pitch.																	

Rivets  $\frac{3}{4}$ -in. diam. at 6-in. pitch.

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 180.  
Safe loads are calculated by the Moncrieff Formula for stanchions of mild steel having both ends flat."

Safe loads for the condition of "both ends fixed" are identical with tabular loads on the heights to left of zigzag line

For other conditions and formulae, see notes commencing page 192.

For explanations of properties, &c., see Part IV.

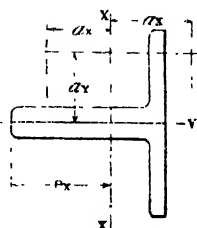
# REDPATH, BROWN & CO., LIMITED.

## COMPOUND STANCHIONS (or STRUTS).

Two Steel Unequal Angles Back to Back.

Short Legs Outstanding.

Dimensions and Properties.



Composed of Two Unequal Angles.	Weight per foot in lbs	Area in square inches.	Distance $e_x$ inches	Radii of Gyration.		Eccentricity Coefficients.	
				Axis Y-Y	Axis X-X	Axis Y-Y	Axis X-X
$7 \times 3\frac{1}{2} \times \frac{5}{8}$	51	14.62	4.40	1.24	2.21	$1+2.26a_y$	$1+0.90a_x$
" $\times \frac{3}{4}$	57	12.31	4.45	1.22	2.22	$1+2.35a_y$	$1+0.90a_x$
" $\times \frac{1}{2}$	35	10.00	4.50	1.20	2.24	$1+2.44a_y$	$1+0.90a_x$
$6 \times 4 \times \frac{5}{8}$	41	11.72	3.98	1.51	1.88	$1+1.74a_y$	$1+1.12a_x$
" $\times \frac{3}{4}$	33	9.50	4.03	1.49	1.90	$1+1.80a_y$	$1+1.12a_x$
$6 \times 3\frac{1}{2} \times \frac{5}{8}$	39	11.10	3.89	1.28	1.89	$1+2.12a_y$	$1+1.09a_x$
" $\times \frac{3}{4}$	32	9.00	3.94	1.26	1.91	$1+2.20a_y$	$1+1.08a_x$
" $\times \frac{1}{2}$	24	6.85	3.99	1.24	1.91	$1+2.28a_y$	$1+1.09a_x$
$6 \times 3 \times \frac{5}{8}$	37	10.47	3.78	1.06	1.89	$1+2.66a_y$	$1+1.06a_x$
" $\times \frac{3}{4}$	30	8.50	3.83	1.04	1.91	$1+2.78a_y$	$1+1.05a_x$
" $\times \frac{1}{2}$	23	6.47	3.88	1.01	1.92	$1+2.91a_y$	$1+1.05a_x$

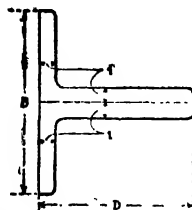
In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

Each weight per foot is for the riveted shaft only. Weight of connections, &c., to be added. Least radii of gyration and relative eccentricity coefficients are printed in prominent type.  $W_e$ =actual eccentric load,  $K$ =relative eccentricity coefficient;  $W_c$ =equivalent concentric value;  $W_c = W_e \times K$ .

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for  $a_y$  and  $a_x$  respectively.

For full explanations of tables, see notes commencing page 192.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND STANCHIONS (or STRUTS).

Two Steel Unequal Angles Back to Back.

Short Legs Outstanding.

Safe Concentric Loads, in Tons.

Ends Flat.

Reference Mark.	Size, D x B inches.	HEIGHTS IN FEET.															
		2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	
17f T	5 x 8	69.8	69.5	69.0	68.4	67.6	66.7	65.7	64.5	63.1	61.7	60.1	58.4	56.6	50.8	44.7	
17e T	"	56.7	56.4	56.0	55.5	54.8	54.1	53.2	52.2	51.1	49.9	48.6	47.1	45.7	40.1	35.2	
17d T	"	43.1	42.9	42.6	42.2	41.4	41.1	40.4	39.6	38.8	37.8	36.8	35.7	34.0	29.6	26.0	
15f T	5 x 6	61.2	60.6	59.7	58.6	57.2	55.6	53.8	51.7	49.5	47.1	44.6	42.9	41.2	35.1	22.1	
15e T	"	49.8	49.3	48.5	47.6	46.4	44.5	43.5	41.7	39.3	37.1	34.7	32.0	29.8			
15d T	"	37.9	37.5	36.9	36.2	35.3	34.2	32.9	31.5	29.8	28.4	26.5	24.9	23.1	18.5	14.5	
11e T	4 x 6	43.2	42.8	42.3	41.5	40.7	39.6	38.4	37.1	35.7	34.2	32.0	29.9	27.8	19.8	17.2	
11d T	"	33.0	32.7	32.3	31.7	31.0	30.2	29.2	28.2	27.0	25.5	23.9	22.6	21.5	16.8	12.6	
7d T	3 x 5	25.5	25.1	24.7	24.1	23.4	22.5	21.6	20.4	18.5	17.7	16.6	15.6	14.6	9.8		
7c T	"	21.5	21.2	20.8	20.3	19.7	18.9	18.1	16.8	15.3	14.1	13.1	12.1	11.1	9.5	8.1	
Rivets $\frac{3}{8}$ -in. diam. at 6-in. pitch.																	

Rivets  $\frac{3}{4}$ -in. diam. at 6-in. pitch.

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160.

Safe loads are calculated by the Moncrieff Formulae for stanchions of mild steel having "both ends flat."

Safe loads for the condition of "both ends fixed" are identical with tabular loads on the heights to left of zigzag line.

For other conditions and formulae, see notes commencing page 192.

Safe loads printed in italics are for heights greater than 40D.

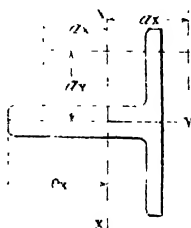
For explanations of properties, &c., see Part IV.

# COMPOUND STANCHIONS (or STRUTS).

Two Steel Unequal Angles Back to Back.

Short Legs Outstanding.

Dimensions and Properties.



Composed of Two Unequal Angles.	Weight per foot in lbs.	Area in square inches	Distance e, inches.	Radii of Gyration.		Eccentricity Coefficients.	
				Axis Y-Y	Axis X-X	Axis Y-Y	Axis X-X
5 x 4 x 5/8	37	10.47	3.39	1.60	1.54	1 + 1.56a <sub>v</sub>	1 + 1.43a <sub>x</sub>
" x 1/2	30 1/2	8.50	3.44	1.58	1.55	1 + 1.61a <sub>v</sub>	1 + 1.43a <sub>x</sub>
" x 3/4	23 1/2	6.47	3.49	1.55	1.57	1 + 1.65a <sub>v</sub>	1 + 1.42a <sub>x</sub>
5 x 3 x 5/8	32 1/2	9.92	3.22	1.13	1.56	1 + 2.36a <sub>v</sub>	1 + 1.32a <sub>x</sub>
" x 1/2	27	7.50	3.26	1.10	1.58	1 + 2.46a <sub>v</sub>	1 + 1.31a <sub>x</sub>
" x 3/4	21	5.72	3.32	1.08	1.59	1 + 2.57a <sub>v</sub>	1 + 1.31a <sub>x</sub>
4 x 3 x 1/2	23	6.50	2.68	1.18	1.24	1 + 2.14a <sub>v</sub>	1 + 1.75a <sub>x</sub>
" x 3/4	17 1/2	4.97	2.73	1.16	1.25	1 + 2.23a <sub>v</sub>	1 + 1.75a <sub>x</sub>
3 x 2 1/2 x 3/4	14	3.84	2.05	1.00	0.92	1 + 2.47a <sub>v</sub>	1 + 2.43a <sub>x</sub>
" x 1/2	12	3.24	2.08	0.99	0.92	1 + 2.53a <sub>v</sub>	1 + 2.37a <sub>x</sub>

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2 1/2 per cent. over this must be allowed. See page 7.

Each weight per foot is for the riveted shaft only. Weight of connections, &c., to be added. Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

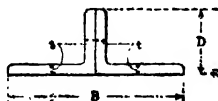
W<sub>e</sub>=actual eccentric load, K=relative eccentricity coefficient; W<sub>ec</sub>=equivalent concentric value; W<sub>c</sub>=W<sub>e</sub> x K.

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for a<sub>v</sub> and a<sub>x</sub> respectively.

For full explanations of tables, see notes commencing page 192.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND STANCHIONS (or STRUTS).



Two Steel Unequal Angles Back to Back.

Long Legs Outstanding.

Safe Concentric Loads, in Tons.  
Ends Flat.

Reference Mark.	Size, D x B inches.	HEIGHTS IN FEET.													
		2	3	4	5	6	7	8	9	10	11	12	13	14	
25g U	3½ x 14	96.7	95.2	93.0	90.0	86.8	82.9	78.5	62.4	50.6	41.8	35.7			
25f U	"	81.6	80.4	78.6	76.2	73.5	70.3	66.6	54.0	43.7	36.2	30.4			
25e U	"	66.2	65.2	63.8	61.9	59.7	57.2	54.3	44.9	36.4	30.1	25.3			
21f U	4 x 12	77.8	77.0	75.9	74.5	72.7	70.6	68.3	65.6	62.6	51.7	43.5	37.0	31.5	
21e U	"	63.1	62.5	61.6	60.5	59.1	57.4	55.5	45.1	42.8	36.0	30.7	26.4		
20f U	3½ x 12	73.5	72.4	70.9	69.0	66.7	64.0	60.9	52.2	42.3	35.0	29.4			
20e U	"	59.6	58.8	57.6	56.1	54.2	52.1	49.6	43.5	35.2	29.1	24.4			
20d U	"	45.3	44.7	43.8	42.7	41.3	39.7	37.9	33.7	27.3	22.6	19.0			
63f U	3 x 12	68.9	67.4	65.3	62.6	59.4	54.3	41.6	32.8	26.6					
63e U	"	56.0	54.8	53.2	51.0	48.5	45.5	43.8	27.5	22.3					
63d U	"	42.6	41.8	40.5	39.0	37.1	34.9	27.2	22.1	17.4					
Rivets ½-in. diam. at 6-in. pitch.															

Rivets ½-in. diam. at 6-in. pitch.

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 180.

Safe loads are calculated by the Moncrieff Formulae for stanchions of mild steel having "both ends flat."

Safe loads for the condition of "both ends fixed" are identical with tabular loads on the heights to left of zigzag line.

For other conditions and formulæ, see notes commencing page 192.

Safe loads printed in italics are for heights greater than 40D.

For explanations of properties, &c., see Part IV.

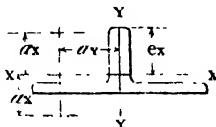
# REDPATH, BROWN & CO., LIMITED.

## COMPOUND STANCHIONS (or STRUTS).

Two Steel Unequal Angles Back to Back.

Long Legs Outstanding.

Dimensions and Properties.



Composed of Two Unequal Angles.	Weight per foot in lbs.	Area in square inches.	Distance $e_x$ inches.	Radii of Gyration.		Eccentricity Coefficients.	
				Axis Y-Y	Axis X-X	Axis Y-Y	Axis X-X
$7 \times 3\frac{1}{2} \times \frac{5}{8}$	50 $\frac{1}{2}$	14.62	2.64	3.11	0.90	$1 + 0.60a_y$	$1 + 3.24a_x$
" $\times \frac{3}{8}$	43	12.34	2.69	3.38	0.91	$1 + 0.61a_y$	$1 + 3.22a_x$
" $\times \frac{1}{2}$	35	10.00	2.74	3.35	0.92	$1 + 0.62a_y$	$1 + 3.20a_x$
$6 \times 4 \times \frac{5}{8}$	40 $\frac{1}{2}$	11.72	2.98	2.76	1.12	$1 + 0.79a_y$	$1 + 2.37a_x$
" $\times \frac{3}{8}$	33	9.50	3.03	2.73	1.13	$1 + 0.81a_y$	$1 + 2.36a_x$
$6 \times 3\frac{1}{2} \times \frac{5}{8}$	38 $\frac{1}{2}$	11.10	2.63	2.83	0.94	$1 + 0.75a_y$	$1 + 2.93a_x$
" $\times \frac{3}{8}$	31 $\frac{1}{2}$	9.00	2.68	2.81	0.96	$1 + 0.76a_y$	$1 + 2.91a_x$
" $\times \frac{1}{2}$	24	6.85	2.73	2.77	0.97	$1 + 0.78a_y$	$1 + 2.90a_x$
$6 \times 3 \times \frac{5}{8}$	36 $\frac{1}{2}$	10.47	2.27	2.92	0.77	$1 + 0.71a_y$	$1 + 3.80a_x$
" $\times \frac{3}{8}$	29 $\frac{1}{2}$	8.50	2.32	2.89	0.78	$1 + 0.72a_y$	$1 + 3.76a_x$
" $\times \frac{1}{2}$	23	6.47	2.37	2.86	0.79	$1 + 0.73a_y$	$1 + 3.74a_x$

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2 $\frac{1}{2}$  per cent. over this must be allowed. See page 7.

Each weight per foot is for the riveted shaft only. Weight of connections, &c., to be added.

Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

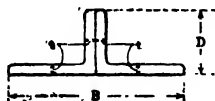
We = actual eccentric load ; K = relative eccentricity coefficient ; We = equivalent concentric value ;  $We = We \times K$ .

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for  $a_y$  and  $a_x$  respectively.

For full explanations of tables, see notes commencing page 192.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND STANCHIONS (or STRUTS).



Two Steel Unequal Angles Back to Back.

Long Legs Outstanding.

Safe Concentric Loads, in Tons.

Ends Flat.

Reference Mark.	Size, D x B inches.	HEIGHTS IN FEET.													
		2	3	4	5	6	7	8	9	10	11	12	13	14	
17f U	4 x 10	69.6	68.9	68.0	66.8	65.3	63.5	61.5	59.3	56.9	49.2	41.4	35.3	30.4	
17e U	"	56.5	56.0	55.2	54.3	53.1	51.7	50.1	48.4	46.4	40.9	34.4	29.3	25.2	
17d U	"	43.0	42.6	42.1	41.3	40.5	39.4	38.2	36.9	35.5	31.8	26.7	22.7	19.6	
15f U	3 x 10	60.7	59.5	57.8	55.6	53.0	50.0	46.9	43.1	38.5	5.2	5.5			
15e U	"	49.4	48.5	47.1	45.4	43.3	40.9	38.3	35.2	31.3					
15d U	"	37.7	37.0	36.0	34.7	33.2	31.4	29.2	26.7	23.8					
11e U	3 x 8	42.9	42.2	41.1	39.7	38.0	36.1	34.1	32.0	29.7	16.7				
11d U	"	32.8	32.3	31.5	30.4	29.2	27.8	26.4	24.9	23.1	15.9	13.1			
7d U	2½ x 6	25.2	24.6	23.7	22.6	21.3	17.6	13.5	10.6						
7c U	"	21.3	20.8	20.0	19.1	18.0	15.1	11.6	9.1						
Rivets ½-in. diam. at 6-in. pitch.															

Rivets ½-in. diam. at 6-in. pitch.

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160. Safe loads are calculated by the Moncrieff Formulae for stanchions of mild steel having "both ends flat."

Safe loads for the condition of "both ends fixed" are identical with tabular loads on the heights to left of zigzag line.

For other conditions and formulae, see notes commencing page 192.

Safe loads printed in italics are for heights greater than 40D.

For explanations of properties, &c., see Part IV.

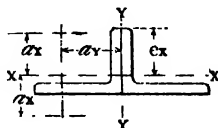
# REDPATH, BROWN & CO., LIMITED.

## COMPOUND STANCHIONS (or STRUTS).

Two Steel Unequal Angles Back to Back.

Long Legs Outstanding.

Dimensions and Properties.



Composed of Two Unequal Angles.	Weight per foot in lbs.	Area in square inches.	Distance $e_x$ inches.	Radii of Gyration.		Eccentricity Coefficients.	
				Axis Y-Y	Axis X-X	Axis Y-Y	Axis X-X
5 × 4 × $\frac{3}{8}$	36½	10.47	2.89	2.22	1.15	1 + 1.01 $a_y$	1 + 2.16 $a_x$
" × $\frac{1}{2}$	29½	8.50	2.94	2.20	1.17	1 + 1.03 $a_y$	1 + 2.15 $a_x$
" × $\frac{3}{4}$	23	6.47	2.99	2.18	1.18	1 + 1.06 $a_y$	1 + 2.14 $a_x$
5 × 3 × $\frac{3}{8}$	32	9.22	2.21	2.37	0.80	1 + 0.89 $a_y$	1 + 3.40 $a_x$
" × $\frac{1}{2}$	26½	7.50	2.26	2.34	0.82	1 + 0.91 $a_y$	1 + 2.99 $a_x$
" × $\frac{3}{4}$	20	5.72	2.31	2.31	0.83	1 + 0.93 $a_y$	1 + 2.97 $a_x$
4 × 3 × $\frac{1}{2}$	23	6.50	2.18	1.80	0.85	1 + 1.23 $a_y$	1 + 3.41 $a_x$
" × $\frac{3}{4}$	17½	4.97	2.23	1.78	0.86	1 + 1.26 $a_y$	1 + 3.40 $a_x$
3 × 2½ × $\frac{3}{8}$	14	3.84	1.80	1.32	0.72	1 + 1.73 $a_y$	1 + 3.37 $a_x$
" × $\frac{1}{2}$	12	3.24	1.83	1.30	0.73	1 + 1.76 $a_y$	1 + 3.35 $a_x$

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

Each weight per foot is for the riveted shaft only. Weight of connections, &c., to be added.

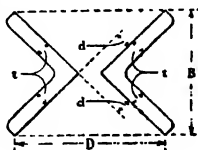
Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

We = actual eccentric load; K = relative eccentricity coefficients; We = equivalent concentric value;  $We = We \times K$ .

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for  $a_y$  and  $a_x$  respectively.

For full explanations of tables, see notes commencing page 192.

# REDPATH, BROWN & CO., LIMITED.

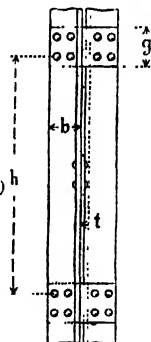


## COMPOUND STANCHIONS (or STRUTS).

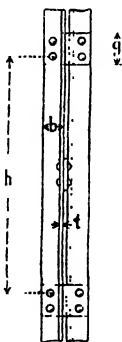
### Two Steel Equal Angles Batted.

Safe Concentric Loads, in Tons.  
Ends Flat.

Reference Mark.	Size, D x B inches.	HEIGHTS IN FEET.															
		2	4	6	8	10	11	12	13	14	15	16	17	18	19	20	
14g V	10 $\frac{3}{8}$ x 8 $\frac{1}{2}$	113	112	111	109	107	106	105	103	102	100	98.9	97.1	95.2	93.2	91.1	
14f V	10 x "	95.1	94.5	93.6	92.3	90.6	89.7	88.6	87.4	86.2	84.9	83.4	82.0	80.4	78.7	77.0	
14e V	9 $\frac{1}{2}$ x "	76.9	76.5	75.7	74.7	73.4	72.6	71.8	70.9	69.9	68.8	67.7	66.5	65.3	63.9	62.6	
13g V	9 x 7 $\frac{1}{2}$	92.6	91.8	90.5	88.6	86.2	84.8	83.3	81.6	79.8	77.9	76.0	72.6	64.7	58.1	52.5	
13f V	8 $\frac{3}{4}$ x "	78.2	77.6	76.5	74.9	72.9	71.7	70.4	69.0	67.6	66.0	64.3	62.0	55.3	49.6	44.8	
13e V	8 $\frac{1}{2}$ x "	63.4	62.9	62.0	60.8	59.2	58.3	57.2	56.1	55.0	53.7	52.4	51.0	45.9	41.2	37.1	
12g V	8 $\frac{5}{8}$ x 6 $\frac{3}{4}$	82.6	81.7	80.2	78.1	75.4	73.9	72.2	70.4	68.4	66.4	64.3	62.0	46.4	41.6	37.6	
12f V	7 $\frac{1}{2}$ x "	69.9	69.1	67.9	66.1	63.9	62.6	61.2	59.7	58.1	56.3	50.3	44.6	39.7	35.7	32.2	
12e V	7 $\frac{1}{8}$ x "	56.7	56.1	55.2	53.8	52.0	51.0	49.9	48.7	47.4	46.0	44.1	43.7	33.1	29.7	26.8	



For 6-in. and 6-in.  
angles.



For 4 $\frac{1}{2}$ -in. to 2-in.  
angles.

The angles forming stanchions or struts of this class are usually secured together with batten plates spaced alternately at right angles to each other.

See opposite page for conventional spacing and proportions.

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160.

Safe loads are calculated by the Moncrieff Formulae for stanchions of mild steel having "both ends flat"

Safe loads for the condition of "both ends fixed" are identical with tabular loads on the heights to left of zigzag line.

For other conditions and formulae, see notes commencing page 192.

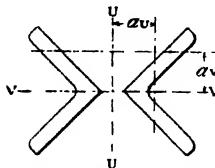
For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND STANCHIONS (or STRUTS).

### Two Steel Equal Angles Battened.

#### Dimensions and Properties.



Composed of Two Equal Angles.	Weight per foot in lbs.	Area in square inches.	Thickness of Batten Plates. Inch.	Radii of Gyration.		Eccentricity Coefficients.	
				Axis V—V	Axis U—U	Axis V—V	Axis U—U
6 × 6 × $\frac{3}{8}$	57½	16.88	$\frac{3}{8}$	2.28	3.23	1+0.82av	1+0.50av
" × 6 × $\frac{5}{16}$	48½	14.22	$\frac{5}{16}$	2.30	3.09	1+0.81av	1+0.52av
" × 6 × $\frac{1}{2}$	39½	11.50	$\frac{1}{2}$	2.32	2.95	1+0.79av	1+0.55av
5 × 5 × $\frac{3}{8}$	47½	13.87	$\frac{3}{8}$	1.88	2.83	1+1.00av	1+0.56av
" × 5 × $\frac{5}{16}$	40	11.72	$\frac{5}{16}$	1.89	2.69	1+0.98av	1+0.59av
" × 5 × $\frac{1}{2}$	32½	9.50	$\frac{1}{2}$	1.92	2.55	1+0.96av	1+0.63av
4½ × 4½ × $\frac{3}{8}$	42½	12.38	$\frac{3}{8}$	1.69	2.63	1+1.12av	1+0.60av
" × 4½ × $\frac{5}{16}$	36	10.47	$\frac{5}{16}$	1.70	2.49	1+1.10av	1+0.64av
" × 4½ × $\frac{1}{2}$	29	8.50	$\frac{1}{2}$	1.72	2.35	1+1.07av	1+0.69av

#### CONVENTIONAL MAXIMUM SPACING AND MINIMUM PROPORTIONS OF BATTEN PLATES FOR CONCENTRIC LOADING (*Am. Ry. Engineering and Maintenance of Way Assoc.*)

Maximum centres of end rivets of batten plates =  $h$  inches.

$h$  = the lesser value of  $\left\{ \begin{array}{l} 10 \text{ times } b \text{ the width of one leg in inches.} \\ 60 \text{ times } t \text{ the angle thickness in inches.} \end{array} \right.$

Minimum width of batten plates =  $g$  inches.

$g$  = the greater value of  $\left\{ \begin{array}{l} b \text{ the width of one leg, or } c \text{ the horizontal centres of rivets, or the} \\ \text{least width suitable for 2 rivets, in inches.} \end{array} \right.$

Rivet diameter =  $\frac{3}{4}$  inch for angles  $\frac{3}{8}$ ,  $\frac{5}{16}$ ,  $\frac{1}{2}$ , and  $\frac{3}{4}$  inch thick.

" " =  $\frac{5}{8}$  " " "  $\frac{7}{8}$  inch thick.

" " =  $\frac{1}{2}$  " " "  $\frac{1}{2}$  and  $\frac{3}{4}$  inch thick.

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of  $\frac{3}{4}$  per cent. over this must be allowed. See page 7.

Each weight per foot is for the shaft only. Weight of batten plates, rivets, base, &c., to be added.

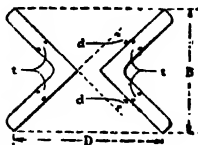
Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

$W$ =actual eccentric load;  $K$ =relative eccentricity coefficient;  $W_c$ =equivalent concentric value;  $W_m = W \times K$ .

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for  $av$  and  $uv$  respectively.

For full explanation of tables, see notes commencing page 182.

REDPATH, BROWN & CO., LIMITED.



**COMPOUND STANCHIONS  
(or STRUTS).**

**Two Steel Equal Angles Battened.**

**Safe Concentric Loads, in Tons.  
Ends Flat.**

Reference Mark.	Size, D x B inches.	HEIGHTS IN FEET.														
		2	3	4	5	6	7	8	9	10	11	12	13	14	15	
11g V	7½ x 5½	72.5	72.1	71.5	70.7	69.8	68.7	67.4	66.0	64.4	62.6	60.7	58.7	56.1	45.4	
11f V	7½ x "	61.4	61.1	60.6	60.0	59.2	58.3	57.2	56.0	54.7	53.3	51.7	50.0	45.1	39.3	
11e V	6½ x "	50.0	49.7	49.3	48.8	48.2	47.5	47.0	46.7	44.7	43.5	42.3	41.0	37.8	32.9	
11d V	6½ x "	38.1	37.9	37.6	37.2	36.8	36.3	35.6	34.9	34.1	33.3	32.4	31.4	29.4	25.6	
10f V	6½ x 5	53.0	52.6	52.1	51.3	50.4	49.4	48.1	46.8	45.3	43.6	41.9	40.1	29.0	25.3	
10e V	6½ x "	43.3	43.0	42.5	41.9	41.2	40.4	39.4	38.3	37.1	35.8	34.3	32.8	24.5	21.3	
10d V	5½ x "	33.1	32.8	32.5	32.1	31.6	30.9	30.2	29.4	28.5	27.6	26.3	25.4	19.3	16.8	
9f V	5½ x 4½	44.6	44.1	43.5	42.6	41.6	40.3	38.9	37.3	34.4	32.8	31.2	29.9	20.3	17.5	
9e V	5½ x "	36.6	36.2	35.6	35.0	34.1	33.2	32.0	30.8	29.2	27.4	26.0	24.3	17.3	14.9	
9d V	5½ x "	28.0	27.8	27.4	26.9	26.3	25.5	24.7	23.8	22.7	21.9	21.1	20.1	13.7	11.8	
9c V	5½ x "	23.6	23.4	23.0	22.6	22.1	21.5	20.8	20.0	19.2	18.4	17.8	17.1	11.8	10.1	
9b V	4½ x "	19.1	18.9	18.7	18.3	17.9	17.4	16.9	16.3	15.6	14.4	13.3	12.3	9.6	8.3	
7e V	4½ x 3½	29.7	29.3	28.6	27.8	26.8	25.6	24.3	22.9	21.6	20.1	18.7	17.1			
7d V	4½ x "	22.9	22.6	22.1	21.5	20.8	19.9	18.9	17.9	16.9	15.9	14.9	13.7	9.0		
7c V	4½ x "	19.3	19.0	18.6	18.1	17.5	16.8	16.0	15.3	14.1	13.1	12.1	11.1			
7b V	4½ x "	15.7	15.5	15.2	14.8	14.3	13.7	13.0	12.3	11.3	10.2	9.2	8.4			
6c V	4½ x 3½	14.9	14.6	14.3	13.8	13.2	12.5	10.8	8.5	6.9	5.7					
6b V	3½ x "	14.0	13.7	13.4	12.9	12.4	11.8	10.2	8.1	6.5	5.4					
5b V	3½ x 2½	12.3	12.0	11.6	11.1	10.5	9.0	6.9	5.4							
5a V	3½ x "	9.5	9.2	8.9	8.5	8.1	7.1	5.4	4.3	3.5						

For sketch, see page 180.

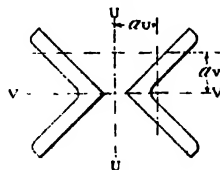
The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160.  
Safe loads are calculated by the Moncrieff Formulae for stanchions of mild steel having "both ends flat."  
Safe loads for the condition of "both ends fixed" are identical with tabular loads on the heights to left of zigzag line.  
For other conditions and formulae, see notes commencing page 182.  
Safe loads printed in italics are for heights greater than 40 ft.  
For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND STANCHIONS (or STRUTS).

Two Steel Equal<sup>o</sup> Angles Battened.

Dimensions and Properties.



Composed of Two Equal Angles.	Weight per foot in lbs.	Area in square inches.	Thickness of Batten Plate. Inch.	Radii of Gyration.		Eccentricity Coefficients.	
				Axis V—V	Axis U—U	Axis V—V	Axis U—U
4 × 4 × $\frac{3}{8}$	37	10.87	$\frac{3}{8}$	1.48	2.44	1+1.28a <sub>v</sub>	1+0.64a <sub>u</sub>
" × $\frac{1}{2}$	31½	9.22	$\frac{1}{2}$	1.50	2.29	1+1.26a <sub>v</sub>	1+0.69a <sub>u</sub>
" × $\frac{5}{8}$	25½	7.50	$\frac{5}{8}$	1.52	2.15	1+1.22a <sub>v</sub>	1+0.74a <sub>u</sub>
" × $\frac{3}{4}$	19½	5.72	$\frac{3}{4}$	1.54	2.01	1+1.19a <sub>v</sub>	1+0.81a <sub>u</sub>
3½ × 3½ × $\frac{3}{8}$	27½	7.97	$\frac{3}{8}$	1.29	2.10	1+1.47a <sub>v</sub>	1+0.74a <sub>u</sub>
" × $\frac{1}{2}$	22½	6.50	$\frac{1}{2}$	1.31	1.95	1+1.43a <sub>v</sub>	1+0.81a <sub>u</sub>
" × $\frac{5}{8}$	17	4.97	$\frac{5}{8}$	1.34	1.81	1+1.38a <sub>v</sub>	1+0.89a <sub>u</sub>
3 × 3 × $\frac{3}{8}$	23	6.72	$\frac{3}{8}$	1.09	1.90	1+1.76a <sub>v</sub>	1+0.81a <sub>u</sub>
" × $\frac{1}{2}$	19	5.50	$\frac{1}{2}$	1.12	1.76	1+1.70a <sub>v</sub>	1+0.89a <sub>u</sub>
" × $\frac{5}{8}$	14½	4.22	$\frac{5}{8}$	1.13	1.61	1+1.64a <sub>v</sub>	1+0.99a <sub>u</sub>
" × $\frac{3}{4}$	12½	3.55	$\frac{3}{4}$	1.15	1.58	1+1.61a <sub>v</sub>	1+1.01a <sub>u</sub>
" × $\frac{1}{2}$	10	2.88	$\frac{1}{2}$	1.15	1.47	1+1.60a <sub>v</sub>	1+1.11a <sub>u</sub>
2½ × 2½ × $\frac{1}{2}$	15½	4.50	$\frac{1}{2}$	0.91	1.56	1+2.12a <sub>v</sub>	1+0.99a <sub>u</sub>
" × $\frac{5}{8}$	12	3.47	$\frac{5}{8}$	0.93	1.41	1+2.02a <sub>v</sub>	1+1.11a <sub>u</sub>
" × $\frac{3}{4}$	10	2.92	$\frac{3}{4}$	0.94	1.38	1+1.97a <sub>v</sub>	1+1.14a <sub>u</sub>
" × $\frac{1}{2}$	8½	2.37	$\frac{1}{2}$	0.95	1.26	1+1.94a <sub>v</sub>	1+1.28a <sub>u</sub>
2¼ × 2¼ × $\frac{5}{8}$	9	2.26	$\frac{5}{8}$	0.84	1.28	1+2.22a <sub>v</sub>	1+1.22a <sub>u</sub>
" × $\frac{3}{4}$	7½	2.12	$\frac{3}{4}$	0.85	1.17	1+2.20a <sub>v</sub>	1+1.37a <sub>u</sub>
2 × 2 × $\frac{1}{2}$	6½	1.88	$\frac{1}{2}$	0.74	1.07	1+2.56a <sub>v</sub>	1+1.48a <sub>u</sub>
" × $\frac{5}{8}$	5	1.44	$\frac{5}{8}$	0.75	1.03	1+2.49a <sub>v</sub>	1+1.54a <sub>u</sub>

For conventional spacing and proportions, see page 181.

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

Each weight per foot is for the shaft only. Weight of batten plates, rivets, base, &c., to be added.

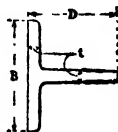
Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

W=actual eccentric load; K=relative eccentricity coefficient; W<sub>e</sub>=equivalent concentric value; W<sub>e</sub>=W×K.

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for a<sub>v</sub> and a<sub>u</sub> respectively.

For full explanations of tables see notes commencing page 182.

# REDPATH, BROWN & CO., LIMITED.



## **STANCHIONS (or STRUTS).** **Steel Tees.**

Safe Concentric Loads, in Tons.  
Ends Flat.

Reference Mark.	Size, B × D × t inches.	HEIGHTS IN FEET.													
		2	3	4	5	6	7	8	9	10	11	12	13	14	
21e W	6 × 4 × $\frac{1}{2}$	31·7	31·4	30·9	30·3	29·6	28·8	27·8	26·8	25·6	21·3	17·9	15·3	13·1	
20e W	6 × 3 × $\frac{1}{2}$	28·1	27·5	26·7	25·6	24·3	22·8	17·5	13·8	11·2					
20d W	" × $\frac{3}{8}$	21·5	21·0	20·4	19·6	18·6	17·5	13·7	10·8	8·7					
19e W	5 × 4 × $\frac{1}{2}$	28·3	28·0	27·6	27·0	26·3	25·5	24·6	23·6	21·3	17·6	14·8	12·6	10·9	
19d W	" × $\frac{3}{8}$	21·6	21·4	21·0	20·6	20·0	19·4	18·7	17·9	15·7	13·0	10·9	9·3	8·0	
17e W	5 × 3 × $\frac{1}{2}$	24·8	24·3	23·6	22·8	21·7	20·5	16·7	13·2	10·7					
17d W	" × $\frac{3}{8}$	18·9	18·6	18·1	17·4	16·7	15·8	13·1	10·3	8·4	6·9				
16e W	4 × 5 × $\frac{1}{2}$	28·1	27·5	26·6	25·5	24·2	22·4	17·1	13·5	11·0					
16d W	" × $\frac{3}{8}$	21·4	20·9	20·2	19·4	18·3	16·4	12·5	9·9	8·0					

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160. Safe loads are calculated by the Moncrieff Formulae for stanchions of mild steel having "both ends flat."

Safe loads for the condition of "both ends fixed" are identical with tabular loads on the heights to left of zigzag line.

For other conditions and formulae, see notes commencing page 102.

Safe loads printed in italics are for heights greater than 40D.

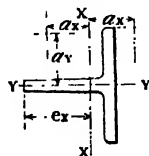
For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## STANCHIONS (or STRUTS).

### Steel Tees.

#### Dimensions and Properties.



Size, B x D x t inches.	Weight per foot in lbs.	Area in square inches.	Distance $e_x$ inches.	Radii of Gyration.		Eccentricity Coefficients.	
				Axis Y-Y	Axis X-X	Axis Y-Y	Axis X-X
$6 \times 4 \times \frac{1}{2}$	16.22	4.771	3.03	1.34	1.13	$1 + 1.66a_y$	$1 + 2.39a_x$
$6 \times 3 \times \frac{1}{2}$	14.53	4.272	2.32	1.42	0.78	$1 + 1.48a_y$	$1 + 3.76a_x$
" $\times \frac{3}{8}$	11.08	3.260	2.37	1.40	0.79	$1 + 1.53a_y$	$1 + 3.75a_x$
$5 \times 4 \times \frac{1}{2}$	14.51	4.268	2.95	1.08	1.16	$1 + 2.13a_y$	$1 + 2.18a_x$
" $\times \frac{3}{8}$	11.07	3.257	3.00	1.06	1.17	$1 + 2.21a_y$	$1 + 2.19a_x$
$5 \times 3 \times \frac{1}{2}$	12.79	3.762	2.26	1.15	0.82	$1 + 1.87a_y$	$1 + 3.37a_x$
" $\times \frac{3}{8}$	9.78	2.875	2.31	1.13	0.83	$1 + 1.94a_y$	$1 + 3.37a_x$
$4 \times 5 \times \frac{1}{2}$	14.50	4.264	3.47	0.78	1.56	$1 + 3.31a_y$	$1 + 1.43a_x$
" $\times \frac{3}{8}$	11.06	3.253	3.53	0.76	1.54	$1 + 3.45a_y$	$1 + 1.48a_x$

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

Each weight per foot is for the shaft only. Weight of connections, &c., to be added.

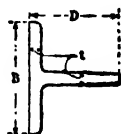
Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

$W_e$  = actual eccentric load;  $K$  = relative eccentricity coefficient;  $W_c$  = equivalent concentric value;  $W_c = W_e \times K$ .

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for  $a_y$  and  $a_x$  respectively.

For full explanations of tables, see notes commencing page 192.

# REDPATH, BROWN & CO., LIMITED.



## STANCHIONS (or STRUTS).

### Steel Tees.

Safe Concentric Loads, in Tons.  
Ends Flat.

Reference Mark.	Size, B x D x t inches.	HEIGHTS IN FEET.									
		2	3	4	5	6	7	8	9	10	11
15e W	4 x 4 x 1/2	24.8	24.3	23.7	22.8	21.8	20.6	17.2	13.6	11.0	9.1
15d W	" x 7/8	18.9	18.6	18.0	17.4	16.6	15.6	12.6	10.0	8.1	
14e W	4 x 3 x 1/2	21.5	21.1	20.6	19.9	19.0	18.1	15.7	12.4	10.0	8.3
14d W	" x 5/8	16.5	16.2	15.8	15.3	14.6	13.9	12.3	9.7	7.9	6.5
13e W	3 1/2 x 3 1/2 x 1/2	21.4	20.0	20.1	19.2	18.1	15.2	11.6	9.2		
13d W	" x 3/4	16.4	15.9	15.4	14.6	13.7	11.1	8.5	6.7		
11e W	3 x 3 x 1/2	18.0	17.4	16.6	15.6	13.2	9.7	7.4			
11d W	" x 3/4	13.8	13.3	12.7	11.8	9.6	7.1	5.4			
8d W	2 1/2 x 2 1/2 x 3/8	11.2	10.7	9.9	8.0	5.6					
8b W	" x 1/2	7.7	7.3	6.7	5.1	3.5					

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160.

Safe loads are calculated by the Moncrieff Formula for stanchions of mild steel having "both ends flat."

Safe loads for the condition of "both ends fixed" are identical with tabular loads on the heights to left of zigzag line.

For other conditions and formulas, see notes commencing page 192.

Safe loads printed in italics are for heights greater than 40D.

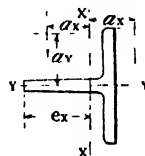
For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## STANCHIONS (or STRUTS).

### Steel Tees.

#### Dimensions and Properties.



Size, B x D x t inches.	Weight per foot in lbs.	Area in square inches.	Distance $e_x$ inches.	Radii of Gyration.		Eccentricity Coefficients.	
				Axis Y-Y	Axis X-X	Axis Y-Y	Axis X-X
4 x 4 x $\frac{1}{2}$	12.78	3.758	2.84	0.83	1.20	$1 + 2.90a_y$	$1 + 1.98a_x$
" x $\frac{3}{4}$	9.77	2.872	2.89	0.81	1.21	$1 + 3.02a_y$	$1 + 1.98a_x$
4 x 3 x $\frac{1}{2}$	11.08	3.260	2.18	0.89	0.85	$1 + 2.51a_y$	$1 + 3.01a_x$
" x $\frac{3}{4}$	8.49	2.498	2.23	0.87	0.86	$1 + 2.61a_y$	$1 + 3.00a_x$
$3\frac{1}{2}$ x $3\frac{1}{2}$ x $\frac{1}{2}$	11.08	3.258	2.46	0.73	1.04	$1 + 3.28a_y$	$1 + 2.27a_x$
" x $\frac{3}{4}$	8.49	2.496	2.51	0.71	1.05	$1 + 3.41a_y$	$1 + 2.27a_x$
3 x 3 x $\frac{1}{2}$	9.38	2.760	2.08	0.63	0.88	$1 + 3.71a_y$	$1 + 2.65a_x$
" x $\frac{3}{4}$	7.21	2.121	2.13	0.62	0.89	$1 + 3.90a_y$	$1 + 2.65a_x$
$2\frac{1}{2}$ x $2\frac{1}{2}$ x $\frac{3}{8}$	5.92	1.741	1.75	0.52	0.74	$1 + 4.61a_y$	$1 + 3.17a_x$
" x $\frac{1}{2}$	4.07	1.197	1.80	0.50	0.75	$1 + 4.96a_y$	$1 + 3.19a_x$

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

Each weight per foot is for the shaft only. Weight of connections, &c., to be added.

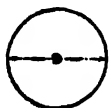
Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

$W_e$  = actual eccentric load;  $K$  = relative eccentricity coefficient;  $W_c$  = equivalent concentric value;  $W_c = W_e \times K$ .

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for  $a_y$  and  $a_x$  respectively.

For full explanations of tables, see notes commencing page 192.

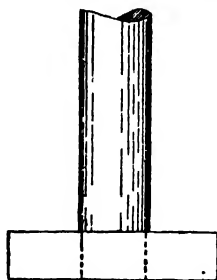
REDPATH, BROWN & CO., LIMITED.



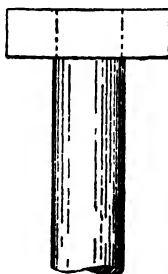
**STANCHIONS (or COLUMNS).**  
**Solid Round Steel.**

Safe Concentric Loads, in Tons.  
Ends Flat.

Reference Mark.	Size, D inches.	HEIGHTS IN FEET.												
		6	8	10	12	14	16	18	20	22	24	26	28	30
23 X	12	749	743	736	726	715	702	687	671	653	634	613	552	481
22 X	11½	688	682	674	664	653	640	626	609	592	572	540	465	405
21 X	11	628	623	615	605	594	581	566	550	533	514	452	390	339
20 X	10½	572	566	558	549	538	525	510	494	476	440	375	323	282
19 X	10	518	512	504	495	484	471	456	440	423	362	309	266	232
18 X	9½	467	461	453	444	432	419	405	389	351	295	251	217	189
17 X	9	418	412	404	395	384	371	357	341	283	238	203	175	152
16 X	8½	372	366	358	349	338	325	311	272	225	189	161	139	
15 X	8	329	323	315	305	294	282	264	214	176	148	126		
14 X	7½	288	282	274	265	254	241	204	165	136	114			



BASE.



CAP.

Bases and Caps are formed of heavy steel slabs, bored out and shrunk on to the accurately machined column ends.

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160.

Safe loads are calculated by the Moncrieff Formulæ for stanchions of mild steel, having "both ends flat."

Safe loads for the condition of "both ends fixed" are identified with tabular loads on the heights to left of zigzag line.

For other conditions and formulæ, see notes commencing page 192

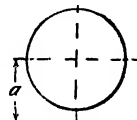
For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## STANCHIONS (or COLUMNS).

### Solid Round Steel.

#### Dimensions and Properties.



Diameter in inches.	Weight per foot in lbs.	Area in square inches.	Radius of Gyrations.	Eccentricity Coefficients.	
				For Semi-diameter.	General.
12	384.6	113.100	3.000	5	1+0.67 <i>a</i>
11½	353.2	103.870	2.875	5	1+0.70 <i>a</i>
11	323.2	95.033	2.750	5	1+0.73 <i>a</i>
10½	294.5	86.590	2.625	5	1+0.76 <i>a</i>
10	267.1	78.540	2.500	5	1+0.80 <i>a</i>
9½	241.0	70.882	2.375	5	1+0.84 <i>a</i>
9	216.3	63.617	2.250	5	1+0.89 <i>a</i>
8½	193.0	56.745	2.125	5	1+0.94 <i>a</i>
8	170.9	50.265	2.000	5	1+1.00 <i>a</i>
7½	150.3	44.179	1.875	5	1+1.07 <i>a</i>

SLABS OF THE UNDERNOTED WIDTHS AND THICKNESSES ARE STOCKED IN LENGTHS OF ABOUT 12 FEET.

Width and Thickness. Inches.	Suitable for Diameters.	Width and Thickness. Inches.	Suitable for Diameters.
18 x 4	10 to 8 inches.	10 x 2	5 to 4 inches.
16 x 3½	8 to 7 "	9 x 1½	4 to 3½ "
14 x 3	7 to 6 "	8 x 1½	3½ to 2½ "
12 x 2½	6 to 5 "	—	—

Above sizes are for concentric loading.

Special calculations are necessary for the design of slab cap plates supporting eccentric loads. See Part IV.

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

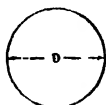
Each weight per foot is for the shaft only. Weight of base, &c., to be added.

$W_e$  = actual eccentric load;  $K$  = relative eccentricity coefficient;  $W_c$  = equivalent concentric value;  $W_c = W_e \times K$ .

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for  $a$ .

For full explanations of tables, see notes commencing page 192.

# REDPATH, BROWN & CO., LIMITED.

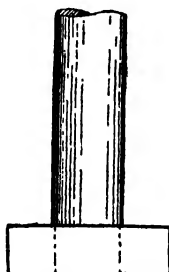


## STANCHIONS (or COLUMNS).

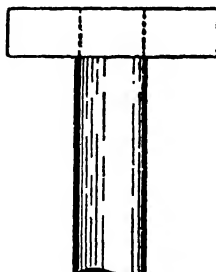
### Solid Round Steel.

Safe Concentric Loads, in Tons.  
Ends Flat.

Reference Mark.	Size, D inches.	HEIGHTS IN FEET.													
		4	6	8	10	11	12	13	14	15	16	18	20	22	
13 X	7	254	250	244	236	231	227	221	216	210	196	155	125	103	
12 X	6½	219	214	208	201	196	191	186	180	165	145	115	93.1		
11 X	6	186	181	175	168	163	158	153	138	120	105	83.4	67.6		
10 X	5½	155	151	145	137	133	128	113	97.4	84.9	74.6	58.9			
9 X	5	128	124	118	110	106	90.6	77.2	66.5	58.0	50.9				
8 X	4½	103	98.7	92.7	85.3	70.7	59.4	50.6	43.7	38.0					
7 X	4	80.7	76.4	70.4	63.4	44.1	37.1	31.6							
6 X	3½	61.0	56.7	48.9	41.3	25.9									
5 X	3	43.9	39.6	26.4	16.9										
4 X	2½	29.4	22.6	12.7											



BASE.



CAP.

Bases and Caps are formed of heavy steel slabs, bored out and shrunk on to the accurately machined column ends.

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160.

Safe loads are calculated by the Moncrieff Formula for stanchions of mild steel having "both ends flat."

Safe loads for the condition of "both ends fixed" are identical with tabular loads on the heights to left of zigzag line.

For other conditions and formulae, see notes commencing page 192.

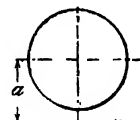
For explanation of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## STANCHIONS (or COLUMNS).

### Solid Round Steel.

#### Dimensions and Properties.



Diameter in inches.	Weight per foot in lbs.	Area in square inches.	Radius of Gyration.	Eccentricity Coefficients.	
				For Semi-diameter.	General.
7	130.9	38.485	1.750	5	1 + 1.15a
6½	112.9	33.183	1.625	5	1 + 1.23a
6	96.13	28.274	1.500	5	1 + 1.34a
5½	80.78	23.758	1.375	5	1 + 1.46a
5	66.76	19.635	1.250	5	1 + 1.60a
4½	54.07	15.904	1.125	5	1 + 1.78a
4	42.72	12.566	1.000	5	1 + 2.00a
3½	32.71	9.621	0.875	5	1 + 2.29a
3	24.03	7.069	0.750	5	1 + 2.67a
2½	16.69	4.909	0.625	5	1 + 3.20a

SLABS OF THE UNDERNOTED WIDTHS AND THICKNESSES ARE STOCKED IN LENGTHS OF ABOUT 12 FEET.

Width and Thickness. Inches.	Suitable for Diameters.	Width and Thickness. Inches.	Suitable for Diameters.
18 x 4	10 to 8 inches.	10 x 2	5 to 4 inches
16 x 3½	8 " 7 "	9 x 1½	4 " 3½ "
14 x 3	7 " 6 "	8 x 1½	3½ " 2½ "
12 x 2½	6 " 5 "		

Above sizes are for concentric loading.

Special calculations are necessary for the design of slab cap plates supporting eccentric loads. See Part IV.

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

Each weight per foot is for the shaft only. Weight of base, &c., to be added.

W<sub>e</sub>=actual eccentric load; K=relative eccentricity coefficient; W<sub>c</sub>=equivalent concentric value; W<sub>c</sub>=W<sub>e</sub>×K.

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for a.

For full explanations of tables, see notes commencing page 192.

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PART II.

Explanations of the Tables.

Pages 122 to 191 inclusive.

See Part IV. for general formulæ, explanations of properties, &c.

Part II.

—

All the tables in this part relate to simple and compound sections, as stanchions, struts or columns.

Arrangement.

The arrangement is similar to that of Part I., precedence being given to the types of stanchions most frequently used.

Compound Stanchions.

A full range of plate thicknesses is given for each joist and channel compound stanchion.

In a series of superimposed stanchions it is convenient and economical to retain the same section of joist or channel throughout, varying the plate areas only, in accordance with the loads.

The tables afford a ready means of selecting suitable types for this purpose.

Latticed Stanchions.

The tabulated safe loads for each latticed stanchion assume efficient bracing between the individual members composing the shaft.

Conventional *minimum* proportions of lattice bars and batten plates for concentric loading are indicated on the tables. Practical considerations will frequently cause the minimum proportions (especially of batten plates) to be increased considerably.

The conventional minimum proportions are not applicable to stanchions under "intentionally" eccentric loading.

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Certain formulæ for the design of lattice bars are noticed in Part IV.

In structural steel work applied to buildings, angle and tee stanchions or struts are usually the compression members of lattice girders or roof trusses.

Angle and Tee  
Stanchions or  
Struts.

The tabulated safe loads for the condition of "both ends flat" are generally applicable to such members, unless each end connection consists of one bolt or one rivet only.

In the latter case refer to the condition of "both ends round," page 199.

Solid round steel stanchions or columns are most useful in positions where considerations of space are of primary importance. For a given load the possible minimum of overall dimensions is attainable with this type.

Solid Rounds.

Particular care should be taken to ensure *concentric* loading on solid round steel stanchions, as the effect of eccentricity is relatively very great.

Various types of stanchions, with suitable designs for bases, caps and connections, are illustrated in Part V.

Details.

All dimensions are stated in inches and all properties in inch units.

Dimensions  
and Properties.

D = depth, B = breadth, and t = thickness.

Overall Sizes.

The composition of compound stanchions is described in the first columns of the right hand pages in the same manner as in Part I.

Composition.

When the plating on each flange exceeds  $\frac{3}{4}$  of an inch, two or more plates may be used to form the total thickness required.

Plate  
Thicknesses.

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**Rivet Pitch.** The Standard rivet pitch for compound stanchions is 6 inches, the diameter being  $\frac{7}{8}$  inch or  $\frac{3}{4}$  inch as indicated on the tables.

**Weights per foot.** Each weight per foot in lbs. of compound plated stanchions, and of double angles back to back, includes an allowance for rivet heads at standard pitch.

Each weight per foot in lbs. is that of the plain or riveted shaft only. The weight of base, cap, connections, lattice bracing, batten plates, extra rivets, &c., requires to be added in estimating the total weight of a complete stanchion.

**Areas.** Each area in square inches is the superficial area of a cross section at right angles to the longitudinal axis.

**Radii of Gyration.** Least and greatest radii of gyration are tabulated for each stanchion.

The least radius of gyration is invariably printed in prominent type.

For each joist and channel and stanchions compounded of these—excepting No. 24 L, page 137, and Nos. 32 M to 29 M inclusive, page 149—the least radius of gyration is about “Axis Y—Y” passing through the centre of gravity of the figure and parallel to the web or webs. The greatest radius of gyration for each of these sections is about “Axis X—X” passing through the centre of gravity of the figure and parallel to the flanges.

The converse applies to Nos. 24 L\* and 32 M to 29 M noted above.

The tabulated radii of gyration for each tee and tee-shaped stanchion formed of two angles back to back are

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also about central axes, but the least radius may be about "Axis Y—Y" parallel to the stalk, or about "Axis X—X" parallel to the table, depending upon the dimensions of the section. Radii of Gyration.

For each simple or latticed angle stanchion, the least radius of gyration is about the major "Axis V—V" of the inertia ellipse. The greatest radius for each of these sections is about the minor "Axis U—U" of the inertia ellipse.

For solid round steel stanchions, all radii of gyration about central axes are identical.

No deduction is made for rivet holes in the calculation of radii of gyration of compound sections. Rivet Holes.

The tabulated safe loads are without exception relative to the least radius of gyration. Tabular Loads.

Dimension "d" in the tables of compound stanchions, pages 136 to 149, and pages 154 to 159, is the spacing of the component joists or channels upon which the tabulated properties are based. Dimension "d."

Any increase or decrease of "d" will therefore increase or decrease the radius of gyration about "Axis Y—Y," and with the exception of No. 24 L, page 137, and Nos. 32 M to 29 M, page 149, will also increase or decrease the tabulated safe loads.

The maximum increase of safe load is reached when "Axis X—X" becomes the axis of least radius, safe loads relative to this axis being constant for all values of "d."

The tabulated safe loads for No. 24 L and Nos. 32 M to 29 M are the maximum for these sections and will be

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decreased when "Axis Y—Y" becomes the axis of least radius.

**Concentric Loading.**

Each tabular load is described as "safe concentric."

This implies that the centre of application of the load or system of loading is, so far as practically possible, coincident with the central vertical axis of the stanchion, or, in other words, that there is no intentional eccentricity of loading.

**Ratio of Slenderness.**

If the height of a stanchion is divided by its least radius of gyration in the same unit dimension (both generally expressed in inches) the quotient is termed the "ratio of slenderness."

$l$  = height of stanchion in inches.

$k$  = least radius of gyration in inches

$\frac{l}{k}$  = ratio of slenderness.

**Limiting Heights.**

In the tables the nearest even height of a stanchion not exceeding that for which the "ratio of slenderness" is equal to 160 is taken as the limiting height for which a safe load is given.

Some authorities prefer to limit the height of a stanchion to the lesser of the two values:

(1) 160 times the least radius of gyration.

(2) 40 times the least overall dimension D or B.

Frequently limit (2) gives a lower height than limit (1).

**Italics.**

For this reason safe loads on all heights greater than the limiting height by (2) are printed in italics in the tables.

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After careful consideration the stanchion formulæ <sup>Stanchion Formulas.</sup> deduced by Mr. J. Mitchell Moncrieff, M.Inst.C.E., M.Amer.Soc.C.E., Newcastle-on-Tyne, have been adopted for this book.

## MONCRIEFF WORKING FORMULÆ

For Stanchions of Mild Steel under Concentric Loading having—

Both ends round.

$$\frac{l}{k} = 100 \sqrt{\left(\frac{21.4}{53.5 - 4.4f}\right) \left(\frac{10.7}{f} - 1.6\right)} \dots \dots (1).$$

Both ends fixed for all values of  $\frac{l}{k}$  and

Both ends flat for values of  $\frac{l}{k}$  not exceeding 106.9.

$$\frac{l}{k} = 200 \sqrt{\left(\frac{21.4}{53.5 - 4.4f}\right) \left(\frac{10.7}{f} - 1.6\right)} \dots \dots (2).$$

Both ends flat for values of  $\frac{l}{k}$  exceeding 106.9.

$$\frac{l}{k} = 200 \sqrt{\frac{21.4 \times 0.4}{5.6f}} \dots \dots \dots (3).$$

$l$  = height of stanchion in inches.

$k$  = radius of gyration in inches.

$\frac{l}{k}$  = ratio of slenderness.

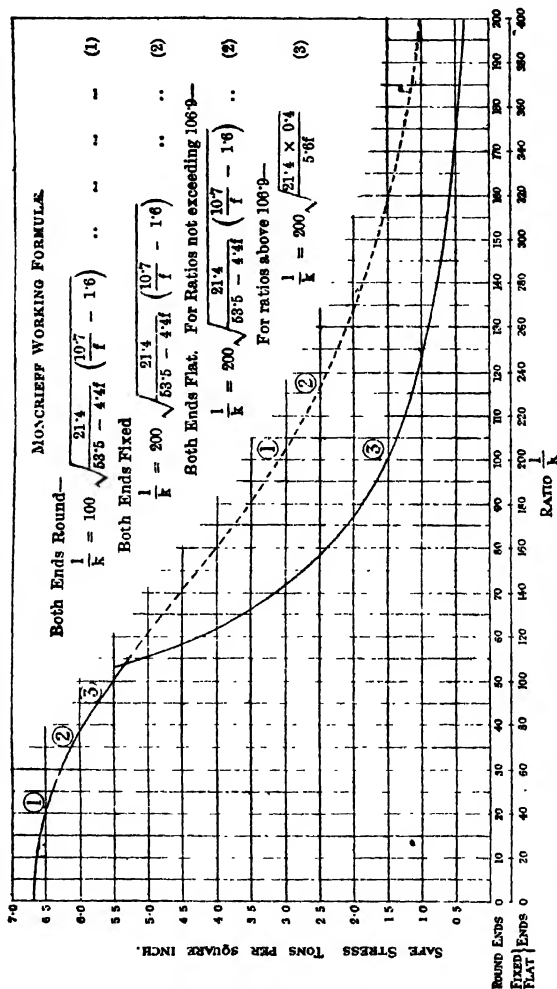
$f$  = average allowable load or working compressive stress per square inch of sectional area in tons per square inch.

The tabular safe loads are by Formulæ (2) and (3) for the condition of "both ends flat."

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## SAFE UNITAL STRESSES IN TONS PER SQUARE INCH BY MONCRIEFF FORMULÆ

### FOR STANCHIONS OF MILD STEEL UNDER CONCENTRIC LOADING.



# REDPATH, BROWN & CO., LIMITED.

Working Stresses, in Tons, per square inch of Section for Stanchions  
of Mild Steel under Concentric Loading by Moncrieff Formula.

$\frac{1}{k}$	(1)	(2)	(3)	$\frac{1}{k}$	(1)	(2)	(3)	$\frac{1}{k}$	(1)	(2)	(3)
4	6.68	6.69		76	4.21	6.01		121	2.85	5.01	4.18
8	6.66	6.68		77	4.16	5.99		122	2.82	4.98	4.11
12	6.62	6.67		78	4.11	5.97		123	2.79	4.96	4.05
16	6.57	6.66		79	4.06	5.95		124	2.76	4.93	3.98
20	6.50	6.65		80	4.01	5.93		125	2.74	4.91	3.92
22	6.46	6.64		81	3.96	5.92		126	2.71	4.88	3.85
24	6.42	6.62		82	3.91	5.90		127	2.68	4.85	3.79
26	6.37	6.61		83	3.86	5.88		128	2.65	4.83	3.74
28	6.32	6.60		84	3.81	5.86		129	2.63	4.80	3.68
30	6.27	6.59		85	3.76	5.84		130	2.60	4.78	3.62
32	6.21	6.57		86	3.71	5.82		132	2.55	4.73	3.51
34	6.14	6.56		87	3.67	5.80		134	2.50	4.67	3.41
36	6.08	6.54		88	3.62	5.78		136	1.96	4.62	3.31
38	6.01	6.52		89	3.57	5.76		138	1.91	4.57	3.21
40	5.93	6.50		90	3.52	5.74		140	1.86	4.52	3.12
42	5.86	6.48		91	3.48	5.71		142	1.82	4.47	3.03
44	5.78	6.46		92	3.43	5.69		144	1.78	4.42	2.95
46	5.69	6.44		93	3.39	5.67		146	1.74	4.36	2.87
48	5.60	6.42		94	3.34	5.65		148	1.70	4.31	2.79
50	5.52	6.40		95	3.30	5.63		150	1.66	4.26	2.72
51	5.47	6.39		96	3.26	5.60		152	1.62	4.21	2.65
52	5.42	6.37		97	3.21	5.58		154	1.58	4.16	2.58
53	5.38	6.36		98	3.17	5.56		156	1.55	4.11	2.51
54	5.33	6.35		99	3.13	5.54		158	1.51	4.06	2.45
55	5.28	6.34		100	3.09	5.52		160	1.48	4.01	2.39
56	5.23	6.32		101	3.05	5.49		162	1.45	3.96	2.33
57	5.18	6.31		102	3.01	5.47		164	1.42	3.91	2.27
58	5.13	6.30		103	2.97	5.45		166	1.39	3.86	2.22
59	5.08	6.28		104	2.93	5.42		168	1.36	3.81	2.17
60	5.03	6.27		105	2.89	5.40		170	1.33	3.76	2.12
61	4.98	6.25		106	2.86	5.38		172	1.30	3.71	2.07
62	4.93	6.24		107	2.82	5.35	5.35	174	1.28	3.67	2.02
63	4.88	6.22		108	2.78	5.33	5.25	176	1.25	3.62	1.97
64	4.83	6.21		109	2.75	5.30	5.15	178	1.23	3.57	1.93
65	4.78	6.19		110	2.71	5.28	5.08	180	1.20	3.52	1.89
66	4.73	6.18		111	2.67	5.26	4.97	182	1.18	3.48	1.85
67	4.67	6.16		112	2.64	5.23	4.88	184	1.16	3.43	1.81
68	4.62	6.14		113	2.61	5.21	4.79	186	1.13	3.39	1.77
69	4.57	6.13		114	2.57	5.18	4.71	188	1.11	3.34	1.73
70	4.52	6.11		115	2.54	5.16	4.65	190	1.09	3.30	1.69
71	4.47	6.10		116	2.51	5.13	4.55	192	1.07	3.26	1.66
72	4.42	6.08		117	2.47	5.11	4.47	194	1.05	3.21	1.62
73	4.36	6.06		118	2.44	5.08	4.40	196	1.03	3.17	1.59
74	4.31	6.04		119	2.41	5.06	4.32	198	1.01	3.13	1.56
75	4.26	6.02		120	2.38	5.03	4.25	200	0.99	3.09	1.53

(1) = Working Stresses for "both ends round" by Formula (1).

(2) = " " " " "both ends fixed" " " (2).

(3) = " " " " "both ends flat" " " (3).

(2) (3) = " " " " "both ends flat" " " (2).

(3) = " " " " "both ends flat" " " (3).

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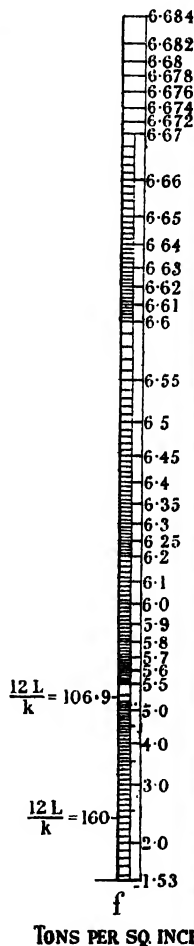
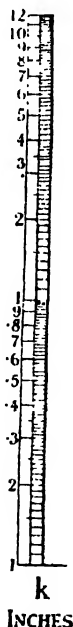
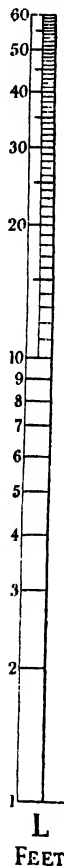
Stanchions.

Moncrieff Formulae.

## ALIGNMENT CHART I.

Lay transparent straight edge across scales—

- At height in feet on scale L. e
- " radius of gyration on scale k.
- Read safe stress on scale f.



$$\frac{12 L}{k} = 106.9$$

$$\frac{12 L}{k} = 160$$

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Stanchions.

Moncrieff Formula.

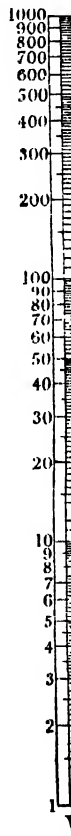
## ALIGNMENT CHART II.

Lay transparent straight edge across scales—

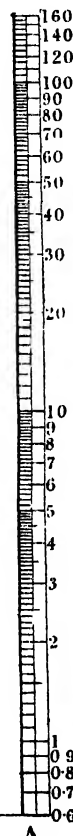
- (a) At safe stress on scale f.
- (b) " area on scale A.
- (c) Read total safe load on scale W.



TONS PER SQ INCH



TONS



SQ INCHES

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**Moncrieff  
Formulae.**

The following notes, pages 202, 203, and 204, are extracted from or based directly on Mr. Moncrieff's paper (see Proceedings Amer. Soc. C.E., Vol. XXVI., 1900).

The underlying principles upon which the reasoning is based are :—

- (1) that a perfectly centred column of perfect material and straightness is an ideal conception seldom or never realised in practice, and
- (2) that the various disturbing influences are practically all capable, as regards their ultimate effect, of being represented by an equivalent eccentricity of loading.

A careful study of no less than 1789 reliable tests of columns was undertaken to determine "the value to be assigned to the equivalent eccentricity applicable to the case of columns under apparently central loading."

The important result of this analysis is that the Moncrieff Formulae include an "equivalent eccentricity" factor to allow for the inherent but practically unobservable defects of the practical column, apart altogether from considerations of intentional eccentricity, treated later in these notes.

**Factors of  
Safety.**

Other special features of the Moncrieff Formulae are that, in preference to the usual method of stanchion design with regard to ultimate strength alone, they ensure :—

- (1) that prescribed maximum allowable fibre stresses will not be exceeded under the working load.

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This condition is complied with by the adoption of a reasonable maximum value for the extreme fibre compressive stress, and

Factors of Safety.

- (2) that in addition a sufficient margin against failure by "elastic instability" is provided in stanchions of longer lengths. The latter coefficient of safety is applied directly to the Modulus of Elasticity of the material.

It is important to note that while the factor of safety against ultimate strength retains a constant value, the use of the additional coefficient of safety against instability in the foregoing manner has an inappreciable influence on the results of the formulæ when applied to short columns, but a gradually increasing effect as the height of the column is increased.

Mr. Moncrieff's theory is founded on the elementary column having "both ends round."

Condition of Ends.

With regard to the condition of "both ends fixed" he states: "In actual practice the true 'fixed-ended' column rarely if ever exists. It is difficult, even in experiments in a testing machine, to comply with the conditions necessary to ensure absolute fixity of ends, and in ordinary construction the difficulty is increased greatly."

In building practice the term "both ends fixed" may, with advantage, be entirely discarded in favour of the term "both ends flat."

"Both ends flat."

Prior to Mr. Moncrieff's investigations no attempt had been made to arrive at a rational basis for the strength of flat-ended columns, although the greater number of tests of columns had been made with this class of end bearing.

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### "Both ends flat."

The usual assumption that "fixed-ended" and "flat-ended" columns act in precisely the same manner is quite erroneous both from a theoretical point of view and from the evidence of actual experiments. The essential distinction is that in the *practical* "flat-ended" column, held in position by pressure of the load and structural bolts or rivets, no tensile stress can be nor is intended to be developed at the ends; in the *theoretical* "fixed-ended" column, on the contrary, a considerable amount of end tensile stress could be developed and safely resisted.

Mr. Moncrieff has shown that in mild steel columns under apparently central loading no end tensile stress is developed provided the ratio of slenderness does not exceed 106·9, and therefore up to this point "fixed-ended" and "flat-ended" columns behave alike. Beyond this point in comparison with the ideal "fixed-ended" column, the strength of the practical "flat-ended" column falls rapidly and should be calculated by Formula (3).

### Zig-zag Line.

The zig-zag lines are inserted in the tables to separate the series of safe loads for ratios of slenderness below and above 106·9.

The safe loads to the left of the lines are identical for "both ends flat" and "both ends fixed." The more rapidly decreasing safe loads for ascending heights to the right of the lines are applicable in particular to the condition of "both ends flat."

### Eccentric Loading.

Eccentric loading is of two descriptions, viz. :—"accidental" and "intentional."

### Accidental Eccentricity.

The effect of "accidental eccentricity" is fully allowed for by the Moncrieff Stanchion Formulæ.

(End of Notes on Moncrieff Formulæ.)

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"Intentional eccentricity" occurs when the perpendicular distance from the centre of application of a load or system of loading to either or both "principal axes" of the stanchion is a quantity measurable by ordinary practical methods.

**Intentional  
Eccentricity.**

In these notes by "eccentricity" will now be understood "intentional eccentricity" as "accidental eccentricity" is not considered further.

**Eccentricity  
(general).**

The measurable distance referred to above is termed the "arm of eccentricity" and is expressed in inches.

**Arm of  
Eccentricity.**

The "principal axes" are "the axis of least radius" and "the axis of greatest radius."

**Principal Axes.**

Loading is said to be "eccentric about the axis" to which the "arm of eccentricity" is perpendicular.

**Loading.**

The tabular eccentricity coefficients are derived from the general formulæ for eccentric loading, for which see Part IV.

**Eccentric  
Loading.**

For each stanchion, eccentricity coefficients relative to both of the "principal axes" of the section are given.

The eccentricity coefficients relative to the "axis of least radius" are printed in prominent type in the tables.

**Prominent  
Type.**

The coefficients in the tables under headings "Axis Y—Y," "Axis V—V" and "Axis U—U" are respectively relative to these "principal axes" and may be termed "axial coefficients."

**Axial  
Coefficients.**

To complete the "axial coefficients" it is only necessary to substitute for  $A_x$ ,  $A_z$ ,  $A_y$ , or  $A_v$ , the actual value in inches of the "arm of eccentricity."

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### Axial Coefficients.

The "axial coefficients" are of general application for any degree of eccentricity, care being taken to select the coefficient having the same reference letters as the axis about which the loading is eccentric.

### Channel Coefficients.

Special note may be made of the "axial coefficients" for each channel stanchion, pages 150 to 153.

As "Axis Y—Y" for this type is not an axis of symmetry, it is necessary to consider on which side of this axis the eccentric loading is placed.

When the "arm of eccentricity" is measured in the same direction as dimension  $e_x$  use coefficient "Axis Y—Y  $e_x$ ," and conversely when the centre of application of the load is on the other side of "Axis Y—Y" use coefficient "Axis Y—Y  $e_y$ ."

### Angle and Tee Coefficients.

For each angle, tee and tee shaped stanchion the "axial coefficients" relative to the asymmetrical axes V—V, U—U, and X—X, take into account the perpendicular distance  $e_y$ ,  $e_x$ , or  $e_z$  to the extreme fibre of the section irrespective of the side of the axis on which the loading occurs. The worst case is thus provided for.

### Web and Flange Coefficients.

In addition to the "axial coefficients" for each joist and channel, and for each stanchion compounded of either of these sections, there are two coefficients for special conditions of eccentric loading, viz.:—"Web" and "Flange," respectively relative to "Axis Y—Y" and "Axis X—X."

These are applicable when the "arm of eccentricity" is identical with the perpendicular distance from "Axis Y—Y" or "Axis X—X" to the outer surface of the web or flange respectively.

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This is usually taken to be the case in good construction when the eccentric load is transmitted by a girder properly connected to a side of a stanchion. **Web and Flange Coefficients.**

By the use of the eccentricity coefficient for the axis about which a load or system of loading is eccentric an "equivalent concentric value" of the load relative to that axis may be obtained. **Equivalent Concentric Load.**

Let  $W_e$  = actual eccentric load in tons.

$K$  = eccentricity coefficient for axis about which  $W_e$  is eccentric.

$W_c$  = equivalent concentric load value in tons for that axis.

Then  $W_c = W_e \times K$ .

It follows from the above that if a tabular concentric load is divided by the eccentricity coefficient printed in prominent type, the maximum safe eccentric load relative to the "axis of least radius" of the section is obtained directly. **Safe Eccentric Load.**

The following examples illustrate the application of the tables to the design of eccentrically loaded stanchions, and also the use of the Alignment Charts, pages 200 and 201. **Examples.**

(A) Loading eccentric about "axis of least radius" only.

*Example 1.*—A stanchion 11 feet high supports an eccentric load of 60 tons, transmitted directly to its web surface by a girder.

Required a suitable section.

Select No. 22 J, page 122, steel joist  $12 \times 6a$  which will support a safe concentric load of 88 tons on 11 feet.

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**Examples.**

Multiply 60 tons, the actual concentric load, by 1.42, the "web" eccentricity coefficient for the section.

The product 85.2 tons is the equivalent concentric load value; therefore the selected stanchion is suitable.

*Example 2.*—A stanchion 20 feet high supports a system of eccentric loading amounting to 150 tons; the "arm of eccentricity" about the "axis of least radius" being 2 ins.

Required a suitable section.

Select No. 142 M, page 144, composed of two steel joists,  $14 \times 6\frac{1}{2}$  and two flange plates  $14" \times \frac{3}{4}"$  which will support a safe concentric load of 300 tons on 20 feet.

Substitute 2 inches, the given "arm of eccentricity," for  $\alpha_r$ , and obtain  $(1 + 0.47 \times 2) = 1.94$  as the "Axis Y—Y" or "axis of least radius" eccentricity coefficient.

Multiply 150 tons, the actual eccentric load, by 1.94.

The product 291 tons is the equivalent concentric load value, therefore the selected stanchion is suitable.

**(B) Loading eccentric about "axis of greatest radius" only.**

*Example 3.*—A stanchion 16 feet high supports an eccentric load of 84 tons transmitted directly to its flange surface by a girder.

Required a suitable section.

Select No. 50 P, page 156, composed of two steel channels  $12 \times 3\frac{1}{2}$  and two flange plates,  $14" \times \frac{1}{2}"$ .

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Note height, 16 feet; area, 33.3 square inches, and *Examples.* greatest radius of gyration "Axis X—X," 5.28 inches.

Transfer these values to the Alignment Charts, pages 200 and 201.

On Chart I. lay a straight edge across the three vertical scales at the height 16 feet on scale L, at the radius of gyration 5.28 inches on scale k, and read safe stress as 6.54 tons per square inch on scale f.

On Chart II. lay straight edge at 6.54 tons on scale f, at the area 33.3 square inches on scale A, and read safe concentric load for "Axis X—X" as 217 tons on scale W.

Divide 217 tons by 2.52, the flange eccentricity coefficient for the section.

The quotient 86 tons is the safe flange eccentric load; therefore the selected stanchion is suitable.

*Example 4.*—A stanchion 16 feet high supports a system of eccentric loading amounting to 165 tons, the "arm of eccentricity" about the "axis of greatest radius" being  $1\frac{1}{2}$  inches.

Required a suitable section.

Select No. 191 K, page 128, composed of 1 steel joist  $15 \times 6$  and 2 flange plates  $12'' \times \frac{5}{8}''$ .

Note height 16 feet, area 32.3 square inches and greatest radius of gyration, "Axis X—X," 6.91 inches.

Transfer these values to the Alignment Charts, pages 200 and 201.

On Chart I., by the method described, read safe stress as 6.61 tons per square inch on scale f.

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### Examples.

On Chart II. read safe concentric load for "Axis X—X" as 213 tons on scale W.

Substitute 1.5, the "arm of eccentricity" for  $a_x$  and obtain  $(1 + 0.17 \times 1.5) = 1.255$  as the "Axis X—X" or "axis of greatest radius" eccentricity coefficient.

Divide 213 tons by 1.255.

The quotient 170 tons is the safe load for an eccentricity of  $1\frac{1}{2}$  inches about "Axis X—X"; therefore the selected stanchion is suitable.

### C.—Loading eccentric about both axes.

Select a stanchion from the tables as in Examples 1 and 2 as if the loading were eccentric about the "axis of least radius" only; then by use of the Alignment Charts and eccentricity coefficient for the "axis of greatest radius" check the section for the load eccentric about "the axis of greatest radius."

### Combined Loading.

If a stanchion supports concentric in addition to eccentric loading, the former, if treated separately, must be added to the equivalent concentric load value to give the total equivalent concentric load.

### Maximum Load.

The actual load eccentric or concentric for "the axis of greatest radius" must in no case exceed the tabular load—calculated for "the axis of least radius."

For notes on the location of the "centre of application" of load systems, see Part IV.

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**PART III.**  
**ROOFS.**

**TYPES OF TRUSSES.**

**STRESSES,**  
**WIND PRESSURE,**  
**DETAILS,**

**Etc.**



## CONTENTS OF PART III.

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# RIDGED ROOF TRUSSES.

Rise of Rafter = 1/4th Span.  
 " Main Tie = 1 in 15.



Fig. 1a

Member.	Stress Coefficients.				Length Coefficient
	Dead Load.		Wind Pressure.		
	Compression.	Tension.	Compression.	Tension.	
A — D	'65		'45		'559
B — E	'65		'70		'559
C — D		'58		'49	'501
C — E		'58		'49	'501
D — E		'08		'07	'217

Rise of Rafter = 1/3rd Span.  
 " Main Tie = 1 in 12.

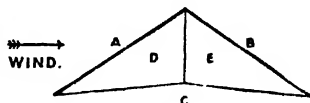


Fig. 1b.

Member	Stress Coefficients.				Length Coefficient.
	Dead Load		Wind Pressure.		
	Compression.	Tension.	Compression.	Tension.	
A - D	.52		.25		.801
B - E	.52		.58		.801
C - D		.43		.29	.502
C - E		.43		.29	.502
D - E		.07		.06	.292

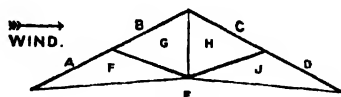


Fig. 2a.

Member.	Stress Coefficients.				Length Coefficient.
	Dead Load.		Wind Pressure.		
	Compression.	Tension.	Compression.	Tension.	
A — F	'97		1'05		'280
D — J	'97		'70		'280
B — G	'64		'57		'280
C — H	'64		'70		'280
F — G	'31		'09		'286
H — J	'31		'00		'286
E — F		'87		1'13	'601
E — J		'87		'48	'601
G — H		'33		'34	'217

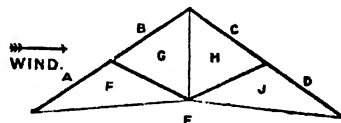


Fig. 2b.

Member	Stress Coefficients.				Length Coefficient.
	Dead Load.		Wind Pressure.		
	Compression.	Tension.	Compression.	Tension.	
A - F	'77		'70		'800
D - J	'77		'58		'800
B - G	'51		'42		'800
C - H	'51		'58		'800
F - G	'24		'58		'280
H - J	'24		'00		'280
E - F		'65		'80	'602
E - J		'05		'29	'602
G - H		'32		'35	'292

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## RIDGED ROOF TRUSSES.

Rise of Rafter = 1/4th Span.

" Main Tie = 1 in 15.

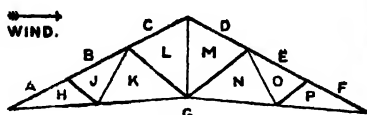


Fig. 3a.

Rise of Rafter = 1/3rd Span.

" Main Tie = 1 in 12.

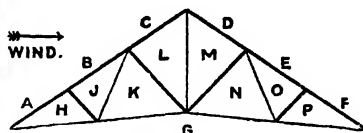


Fig. 3b.

Member.	Stress Coefficients.				Length Coefficient.	Member.	Stress Coefficients.				Length Coefficient.
	Dead Load.		Wind Pressure.				Dead Load.		Wind Pressure.		
	Compression.	Tension	Compression.	Tension			Compression.	Tension.	Compression.	Tension.	
A	1.08		1.24		.188	A	.86		.86		.200
F	1.08		.70		.188	F	.86		.58		.200
B	.94		1.10		.188	B	.74		.80		.200
E	.94		.70		.188	E	.74		.58		.200
C	.66		.62		.188	C	.61		.47		.200
D	.66		.70		.188	D	.61		.58		.200
H	.17		.37		.107	H	.14		.34		.123
O	.17		.00		.107	O	.14		.00		.123
K	.25		.55		.214	K	.21		.51		.246
M	.25		.00		.214	M	.21		.00		.246
G		.96		1.35	.251	G		.72		.98	.251
G		.96		.48	.251	G		.72		.29	.251
G		.77		.92	.251	G		.57		.63	.251
G		.77		.48	.251	G		.57		.29	.251
J		.13		.80	.171	J		.12		.80	.218
N		.13		.00	.171	N		.12		.00	.218
L		.41		.44	.217	L		.40		.45	.292

To find the total stress in any member due to dead load and wind pressure:—

Let L = Span between the points of intersection of the rafter and main tie.

"  $W_D$  = Total dead load on one truss including its own weight.

"  $W_r$  = Total normal wind pressure acting on one side of one truss.

" P = Total stress required.

Then  $P = (W_D \times \text{dead load coefficient}) + (W_r \times \text{wind pressure coefficient})$ .

To find the length of any member between points of intersection:—

Multiply the span by the length coefficient for the member required.

The members are lettered according to Bow's notation. See notes page 220.

Prominent type indicates a greater wind pressure stress on the lee member than on the windward member.

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## RIDGED ROOF TRUSSES.

Rise of Rafter = 1/4th Span.  
 " Main Tie = 1 in 15.

WIND.

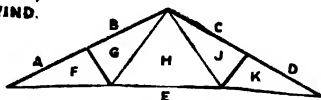


Fig. 4a.

Member.	Stress Coefficients.				Length Coefficient.
	Dead Load.		Wind Pressure		
	Compression.	Tension	Compression.	Tension.	
A - F	'97		1'05		'279
D - K	'97		'70		'279
B - G	'85		1'05		'279
C - J	'85		'70		'279
F - G	'22		'50		'117
J - K	'22		'00		'117
E - F		'87		1'13	'302
E - K		'87		'48	'302
E - H		'54		'46	'895
G - H		'34		'69	'302
H - J		'34		'04	'302

Rise of Rafter = 1/3rd Span.  
 " Main Tie = 1 in 12.

WIND.

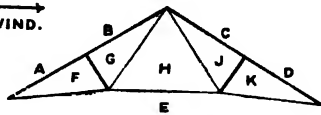


Fig. 4b.

Member.	Stress Coefficients.				Length Coefficient.
	Dead Load.		Wind Pressure.		
	Com- pression.	Tension.	Com- pression.	Tension.	
A — F	'77		'70		'300
D — K	'77		'58		'300
B — G	'63		'70		'300
C — J	'63		'58		'300
F — G	'21		'50		'166
J — K	'21		'00		'166
E — F		'65		'80	'343
E — K		'65		'29	'343
E — H		'41		'28	'317
G — H		'26		'54	'343
H — J		'26		'03	'343

To find the total stress in any member due to dead load and wind pressure:—

Let L = span between the points of intersection of the rafter and main tie.

"  $W_D$  = total dead load on one truss including its own weight.

"  $W_F$  = total normal wind pressure acting on one side of one truss.

" P = total stress required.

Then  $P = (W_D \times \text{dead load coefficient}) + (W_F \times \text{wind pressure coefficient})$ .

To find the length of any member between points of intersection:—

Multiply the span by the length coefficient for the member required.

The members are lettered according to Bow's notation. See notes page 220.

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## RIDGED ROOF TRUSSES.

Rise of Rafter = 1/4th Span.

" Main Tie = 1 in 15.

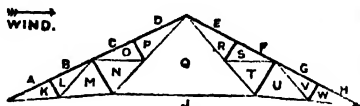


Fig. 5a

Rise of Rafter = 1/3rd Span.

" Main Tie = 1 in 12.

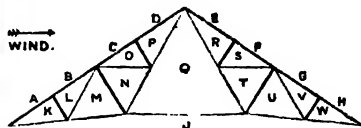


Fig. 5b.

Member.	Stress Coefficients.				Length Coefficient.
	Dead Load.		Wind Pressure.		
	Compression	Tension.	Compression	Tension.	
A-K	1.18		1.84		.140
H-W	1.13		.70		.140
L	1.07		1.34		.140
G-V	1.07		.70		.140
O	1.02		1.34		.140
F-S	1.02		.70		.140
D-P	.96		1.34		.140
E-R	.96		.70		.140
M-N	.22		.50		.117
T-U	.22		.00		.117
K-O	.11		.56		.059
V-W	.11		.00		.059
R-S					.059
J-K		1.01		1.46	.152
J-W		1.01		.48	.152
M		.87		1.13	.152
U		.87		.48	.152
Q		.54		.45	.305
N		.34		.69	.152
T		.34		.04	.152
P		.49		1.01	.152
R		.49		.04	.152
L		.15		.32	.152
N		.15		.00	.152
U					.152
S					.152

Member	Stress Coefficients.				Length Coefficient.
	Dead Load		Wind Pressure.		
	Compression	Tension.	Compression	Tension.	
A-K	.90		.93		.150
H-W	.90		.58		.150
B-L	.83		.93		.150
G-V	.83		.58		.150
O	.76		.93		.150
F-S	.76		.58		.150
D-P	.69		.93		.150
E-R	.69		.58		.150
M-N	.21		.50		.166
T-U	.21		.00		.166
K-O	.10		.25		.083
V-W	.10		.00		.083
R-S					.083
J-K		.75		1.06	.171
J-W		.75		.29	.171
M		.64		.81	.171
U		.64		.29	.171
Q		.41		.28	.317
N		.26		.55	.171
T		.26		.03	.171
P		.36		.30	.171
R		.36		.03	.171
L	}	.11		.26	.171
N		.11		.00	.171
S					

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## RIDGED ROOF TRUSSES.

Rise of Rafter =  $\frac{1}{4}$ th Span.  
 " Main Tie = 1 in 15.

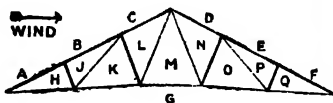


Fig. 6a.

Rise of Rafter =  $\frac{1}{3}$ rd Span.  
 " Main Tie = 1 in 12.

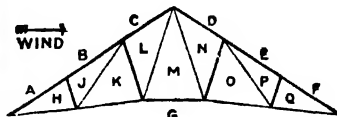


Fig. 6b.

Member		Stress Coefficients.				Length Coefficient
		Dead Load.		Wind Pressure.		
Compression.	Tension	Compression.	Tension			
A - H	1.08		1.24		.186	
F - Q	1.08		.70		.186	
B - J	1.00		1.24		.186	
E - P	1.00		.70		.186	
C - L	.75		.85		.186	
D - N	.75		.70		.186	
H - J	.15		.33		.078	
P - Q	.15		.00		.078	
K - L	.22		.60		.155	
N - O	.22		.00		.155	
G - H		.96		1.35	.200	
G - Q		.96		.43	.200	
G - K		.77		.02	.200	
G - O		.77		.48	.200	
G - M		.56		.47	.200	
J - K		.19		.43	.203	
O - P		.19		.00	.203	
L - M		.27		.55	.245	
M - N		.27		.04	.245	

Member.		Stress Coefficients.				Length Coefficient.
		Dead Load.		Wind Pressure.		
Compression.	Tension.	Compression.	Tension.			
A - H	.86		.86		.200	
F - Q	.86		.58		.200	
B - J	.81		.94		.200	
E - P	.81		.58		.200	
C - L	.60		.68		.200	
D - N	.60		.58		.200	
H - J	.14		.34		.100	
P - Q	.14		.00		.100	
K - L	.22		.62		.200	
N - O	.22		.00		.200	
G - H		.72		.98	.201	
G - Q		.72		.29	.201	
G - K		.58		.63	.201	
G - O		.58		.29	.201	
G - M		.42		.28	.201	
J - K		.18		.42	.245	
O - P		.18		.00	.245	
L - M		.28		.67	.317	
M - N		.28		.08	.317	

To find the total stress in any member due to dead load and wind pressure :—  
 Let L = Span between the points of intersection of the rafter and main tie.  
 "  $W_d$  = Total dead load on one truss including its own weight.  
 "  $W_p$  = Total normal wind pressure acting on one side of one truss.  
 " P = Total stress required.  
 Then  $P = (W_d \times \text{dead load coefficient}) + (W_p \times \text{wind pressure coefficient})$ .  
 To find the length of any member between points of intersection :—  
 Multiply the span by the length coefficient for the member required.  
 The members are lettered according to Bow's notation. See notes, page 220.

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## RIDGED ROOF TRUSSES.

Rise of Rafter = 1/4th Span.  
" Main Tie = 1 in 15.

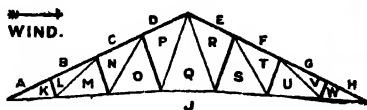


Fig. 7a.

Rise of Rafter = 1/3rd Span.  
" Main Tie = 1 in 12.

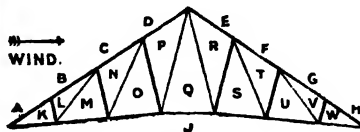


Fig. 7b.

Member.	Stress Coefficients.				Length Coefficient.
	Dead Load.		Wind Pressure.		
	Compression.	Tension.	Compression.	Tension.	
A-K	1.13		1.34		140
H-W	1.13		.70		140
B-L	1.09		1.38		140
G-V	1.09		.70		140
C-N	.90		1.09		140
F-T	.90		.70		140
D-P	.72		.82		140
E-R	.72		.70		140
K-L	.11		.25		.056
V-W	.11		.00		.056
M-N	.17		.38		.112
T-U	.17		.00		.112
O-P	.22		.51		.168
R-S	.22		.00		.168
J-K		1.01		1.46	143
J-W		1.01		.48	143
J-M		.87		1.13	143
J-U		.87		.48	143
J-O		.72		.81	143
J-S		.72		.48	143
J-Q		.57		.47	143
L-M		.16		.36	158
U-V		.16		.00	158
N-O		.19		.43	191
S-T		.19		.00	191
P-Q		.27		.56	233
Q-R		.27		.03	233

Member.	Stress Coefficients.				Length Coefficient.
	Dead Load.		Wind Pressure.		
	Compression.	Tension.	Compression.	Tension.	
A-K	.90		.93		150
H-W	.90		.58		150
B-L	.87		1.02		150
G-V	.87		.58		150
C-N	.72		.84		150
F-T	.72		.58		150
D-P	.58		.66		150
E-R	.58		.58		150
K-L	.11		.27		.074
V-W	.11		.00		.074
M-N	.17		.40		147
T-U	.17		.00		147
O-P	.22		.53		221
R-S	.22		.00		221
J-K		.75		1.06	148
J-W		.75		.29	143
J-M		.64		.81	148
J-U		.64		.29	143
J-O		.54		.55	143
J-S		.54		.29	143
J-Q		.42		.28	143
L-M		.14		.34	189
U-V		.14		.00	189
N-O		.18		.44	244
S-T		.18		.00	244
P-Q		.27		.58	307
Q-R		.27		.03	307

**RIDGED ROOF TRUSSES.**

**PART III.**

**Explanations of Tables of Coefficients.**

Pages 214 to 219.

In roof trusses of the same type as regards the angle of slope of the rafters, and the number and position of the members, *but of any spans*, the stresses in the members due to dead load and wind pressure, and the lengths of the members between points of intersection, are directly proportional to the spans.

**Stress  
Coefficients.**

The tables of coefficients for the various types, figs. 1a to 7b, are calculated on the following assumptions:—

- (1). Both shoes fixed, hence vertical reactions due to dead load, and angular reactions due to wind pressure.
- (2). Unit span between the points of intersection of the rafters and main ties.
- (3). Unit dead load comprising the weights of the truss, purlins and roof covering, uniformly distributed over both rafters and acting in a vertical direction.
- (4). Unit wind pressure uniformly distributed over one rafter and acting in a direction normal or at right angles to its surface.
- (5). Purlins placed over the points of intersection of the various members with the rafters.

## RIDGED ROOF TRUSSES.

Each stress coefficient, therefore, is the value of the compressive or tensile stress in the relative member due to dead load or wind pressure for unit loading on unit span. **Stress Coefficients.**

Similarly, each length coefficient is the length between points of intersection of the relative member for unit span. **Length Coefficients.**

It follows that for any span, any roof covering and any wind pressure required or specified, if the proper tabular stress coefficients for a member are respectively multiplied by the total dead load and total wind pressure acting on the truss, the sum of the two products so ascertained is the total stress in the member, either compression or tension, as the case may be. **Total Stress.**

To find the total stress in any member, due to dead load and wind pressure combined :— **Application of Stress Coefficients.**

$L$  = span between the points of intersection of the rafters and main ties.

$W_D$  = total dead load on one truss, comprising the weights of the truss, purlins and roof coverings.

$W_P$  = total normal wind pressure acting on one side of one truss.

$P$  = total combined stress required.

Then  $P = (W_D \times \text{dead load coefficient}) + (W_P \times \text{wind pressure coefficient}).$

## RIDGED ROOF TRUSSES.

### Application of Length Coefficients.

To find the length of any member between the points of intersection :—

Multiply the span  $L$  by the length coefficient for the member required.

### Total Dead Load.

To calculate  $W_D$ , the total dead load in lbs. on one truss.

$L$  = span of truss in feet.

$C$  = distance of trusses apart, centre to centre, in feet.

$T$  = approximate weight in lbs. of one truss.

$q$  = weight in lbs. of purlins and covering per square foot of roof surface.

$Q$  = total weight in lbs. of purlins and roof covering supported by one truss.

### Weight of Truss.

The following empirical formula (Merriman) gives the approximate weight of a steel truss :—

$$T = \frac{3}{4} C L \left( 1 + \frac{L}{10} \right)$$

### Weight of Purlins, &c.

Total weight of purlins and roof covering :—

$$Q \begin{cases} = q \times C \times 1.12 L & \text{for rise} = \frac{1}{4} \text{th span.} \\ = q \times C \times 1.2 L & \text{" " = } \frac{1}{3} \text{rd "} \end{cases}$$

### Total Dead Load.

Total dead load,  $W_D = T + Q$ .

### Wind Pressure.

To calculate  $W_P$ , the total wind pressure in lbs. on one truss :  $p_n$  = normal wind pressure in lbs. per square foot of roof surface. See Alignment Chart, page 227.

$$W_P \begin{cases} = p_n \times C \times 0.56 L & \text{for rise} = \frac{1}{4} \text{th span.} \\ = p_n \times C \times 0.6 L & \text{" " = } \frac{1}{3} \text{rd "} \end{cases}$$

## RIDGED ROOF TRUSSES.

In the diagrams Figs. 1a to 7b compression members **Diagrams.** are indicated by heavy lines, and tension members by light lines.

The diagrams are lettered according to Bow's notation, each member being designated by the letters on each side of it.

In order to show the distribution of the stresses due to the wind pressure acting on one side of a truss only, coefficients are given for the complete truss, the horizontal direction of the wind being indicated on each diagram by an arrow.

As the wind may act alternately on either side of a truss, the corresponding members on the windward and lee sides must each be designed for the greater stresses.

The greater stresses generally occur on the windward side, but the exception of the king rod trusses, Figs. 1a to 3b, may be noted. In each of these trusses the rafter or portion of the rafter next to the apex on the lee side has a greater compressive stress due to wind pressure than the corresponding windward member. These greater stress coefficients for the lee side are printed in prominent type.

For roof trusses of spans up to 60 feet the rafters **Design.** are usually designed for the maximum stress in the portion next the shoe, the required section being continued to the apex in one length.

## RIDGED ROOF TRUSSES.

### Design.

Special calculations should be made if the span is greater than 60 feet.

Should the purlins not be placed over the points of intersection of the various members with the rafters, the tabular stress coefficients are not applicable, as non-uniform distribution of the loading may result, and bending stresses are produced in the rafters.

### Approximate Weights of Roof Coverings.

	Per Sup. Foot.
Asphalte. For each $\frac{1}{2}$ inch of thickness, ...	6 to $6\frac{1}{2}$ lbs.
Boarding. 1 inch thick, ... ..	3 to $3\frac{1}{2}$ "
Galvanized Corrugated Sheeting, including	
taps and bolts. 18 gauge, ...	3 "
" 20 " ... ..	$2\frac{1}{2}$ "
" 22 " .. ..	2 "
Glass. $\frac{1}{4}$ inch thick, ... ..	$3\frac{1}{2}$ "
Glazing bars (metal), ... ..	$1\frac{1}{2}$ to 3 "
(The lower weight is for purlins spaced about 6 feet apart, and the higher weight for purlins spaced about 10 feet apart.)	
Lead, laid complete including rolls, ...	$5\frac{1}{2}$ to $8\frac{1}{2}$ lbs.
(The lower weight is for 5 lb. lead and the higher weight for 7 lb. lead.)	
Plaster ceiling, 1 inch thick, ... .	9 lbs.
Slates, including nails, ... ..	8 to $8\frac{1}{2}$ "
Steel purlins, ... ..	$1\frac{1}{2}$ to 4 "

Weights of the actual sections of steel purlins should be calculated.

## WIND PRESSURE.

Experiments conducted at the British National Physical Laboratory and also at the Eiffel Tower, Paris, show that the wind pressure per square foot on square flat surfaces from 10 to 100 square feet in extent is 0.0032 times the square of the wind velocity in miles per hour.

Relation  
between  
Pressure and  
Velocity.

In the United States of America the highest wind velocity observed for a period of five minutes was 102 miles per hour at St. Paul, Minnesota (*U.S. Weather Bureau Records*).

Highest  
Recorded  
Velocity.

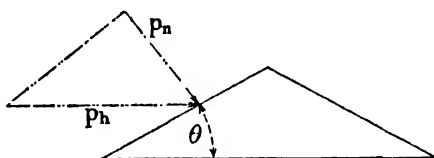
By the Beaufort Scale, a velocity of 56 miles per hour constitutes a strong gale.

From the above it appears that the maximum horizontal wind pressure of 40 lbs. per square foot of exposed vertical surface, as usually assumed in this country, is ample. This pressure corresponds approximately to a velocity of 112 miles per hour.

As fluid pressure is always exerted at right angles to the surface upon which it acts, the corresponding normal pressure calculated by the following formula, or, more conveniently taken from the Alignment Chart, is employed to determine the forces due to wind acting upon the inclined surface of a roof.

Normal Wind  
Pressure.

**WIND PRESSURE—ALIGNMENT CHART.**



$\theta$  = inclination of roof to the horizontal in degrees.

$p_h$  = assumed or specified intensity of horizontal wind pressure in lbs. per square foot of projected vertical surface.

$p_n$  = by formula, intensity of wind pressure in lbs. per square foot of sloping surface, acting normally or at right angles to the slope.

$$p_n = p_h \frac{2 \sin. \theta}{1 + \sin.^2 \theta}$$

This formula which is considered to be the most reliable for the determination of  $p_n$  is due to Duchemin.

**Alignment Chart.**

For any value of  $p_h$  (horizontal wind pressure) between 30 and 50 lbs. per square foot, the Alignment Chart on opposite page gives, without calculation, the value of  $p_n$  (the Duchemin normal component) for any angle of inclination between  $5^\circ$  and  $60^\circ$  with the horizontal. Rises corresponding to  $\frac{1}{3}$ rd,  $\frac{1}{4}$ th, and  $\frac{1}{5}$ th of the span are clearly indicated.

Lay straight-edge across the three vertical scales:—

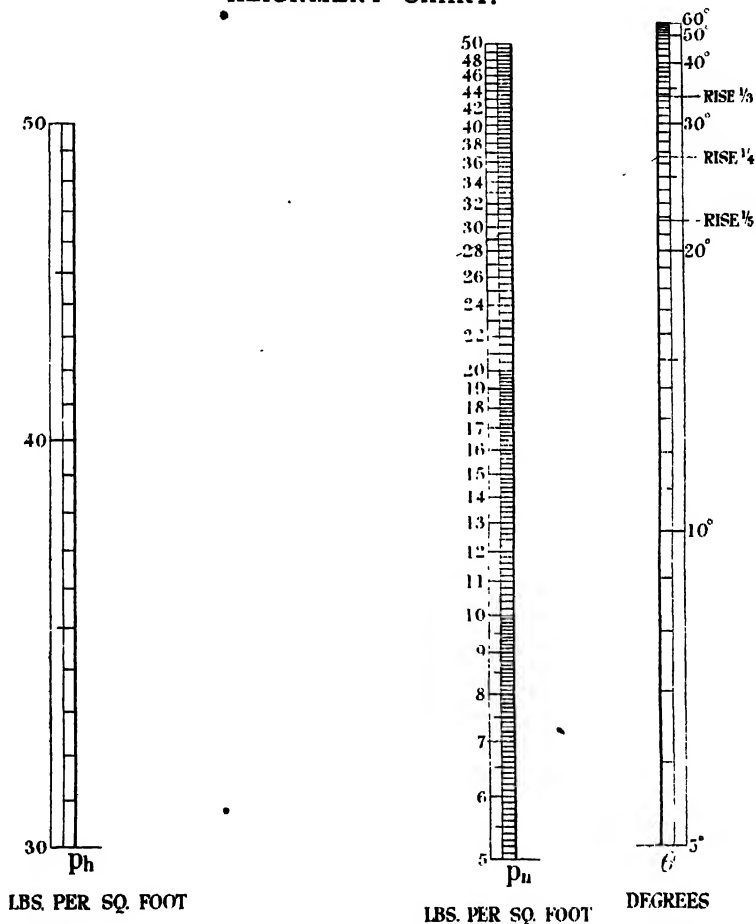
- (1). At horizontal wind pressure value on scale  $p_h$ .
- (2). At angle of inclination or rise on scale  $\theta$ .
- (3). Read value of normal component of wind pressure in lbs. per square foot on scale  $p_n$ .

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### Normal Wind Pressures.

### Duchemin Formula.

### ALIGNMENT CHART.



## RIDGED ROOF TRUSSES.

**Example.**

The following example illustrates the application of the Strength and Length coefficients, the formulæ, and the Alignment Chart:—

Span of Roof Truss,  $L = 30$  feet.

Centres of Trusses,  $C = 10$  "

Rise =  $\frac{1}{4}$ th span.

Type, Fig. 3a.

Covering: 18 gauge galvanized corrugated sheeting on steel purlins.

Horizontal Wind Pressure,  $p_h = 40$  lbs. per square foot.

*Dead Load  $W_D$ .*

$T$  = Approximate weight of truss in lbs.

$$= \frac{3 \times 10 \times 30}{4} \left(1 + \frac{30}{10}\right) = 900 \text{ lbs.}$$

Purlins, cleats and bolts = 2 lbs. per super foot.

Sheeting and fittings = 3 " " " "

$$q = \frac{5}{1} \text{ " " " "}$$

$Q$  = Approximate weight of purlins and covering supported by one truss.

$$= 5 \times 10 \times 1.12 \times 30 = 1680 \text{ lbs.}$$

$$\therefore W_D = T + Q = 900 + 1680 = 2580 \text{ lbs.}$$

*Wind Pressure  $W_P$ —(see Alignment Chart, p. 227).*

With straight-edge join 40 lbs. on scale  $p_h$  with rise  $\frac{1}{4}$ th on scale  $\theta$  degrees.

On centre scale  $p_h$  read 29.8 lbs. as the value of the

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## RIDGED ROOF TRUSSES.

normal wind pressure per square foot of exposed roof surface. **Example.**

(This operation is indicated on the chart by a dotted line).

$$W_P = 29.8 \times 10 \times 0.56 \times 30 = 5006 \text{ lbs.}$$

*Stresses in Members.*—(See Tables, pages 214 to 219).

Member.	Dead Load.		Wind Pressure.				Total Stress in Tons.		Length.	
	Dead Load Stress Coeff.	Stress in Tons = Coeff. $\times$ 2580		Wind Pressure Stress Coeff.	Stress in Tons = Coeff. $\times$ 5006				Length. Coeff.	Length in Feet = Coeff. $\times$ L.
		Compression.	Tension		Compression.	Tension				
A—H	1.080	1.24		1.251	2.80		4.04		186	5.6
H—J	.165	.19		.368	.84		1.01		107	3.2
K—L	.248	.28		.561	1.23		1.51		213	6.4
G—H	.968		1.12	1.356		3.03		4.15		
J—K	.182		.15	.296		.66		.81		
L—M	.412		.47	.437		.97		1.44		

The stresses in this example are calculated for the principal members on the windward side only, as in practice members B-J, C-L, D-M, E-O and F-P would be made of the same section as A-H the portion of the rafter having maximum stress, and similarly G-K, G-N, and G-P would be made of the same section as G-H. For designing it is only necessary to know the lengths of the compression members between their points of intersection.

The minimum size of angle or tee used in roof trusses should be  $2'' \times 2'' \times \frac{1}{4}''$  and the minimum size of flat,  $2'' \times \frac{1}{4}''$ . In designing ties allow for loss of area due to holing.

**Minimum Sections.**

## RIDGED ROOF TRUSSES.

**References.** For detailed weights of galvanized corrugated sheets, sizes of corrugations, &c., - - - See Part IV.  
For angles and tees as purlins, - - - " " I.  
" " " struts (flat ends), " " II.  
" " " " (free " ), See notes, Part II.  
" typical details, - - - - See " V.

**Gutters and Downpipes.** Definite reliable data for proportioning gutters and downpipes is lacking. The recommendations, &c., of some authorities are noted as follows:—

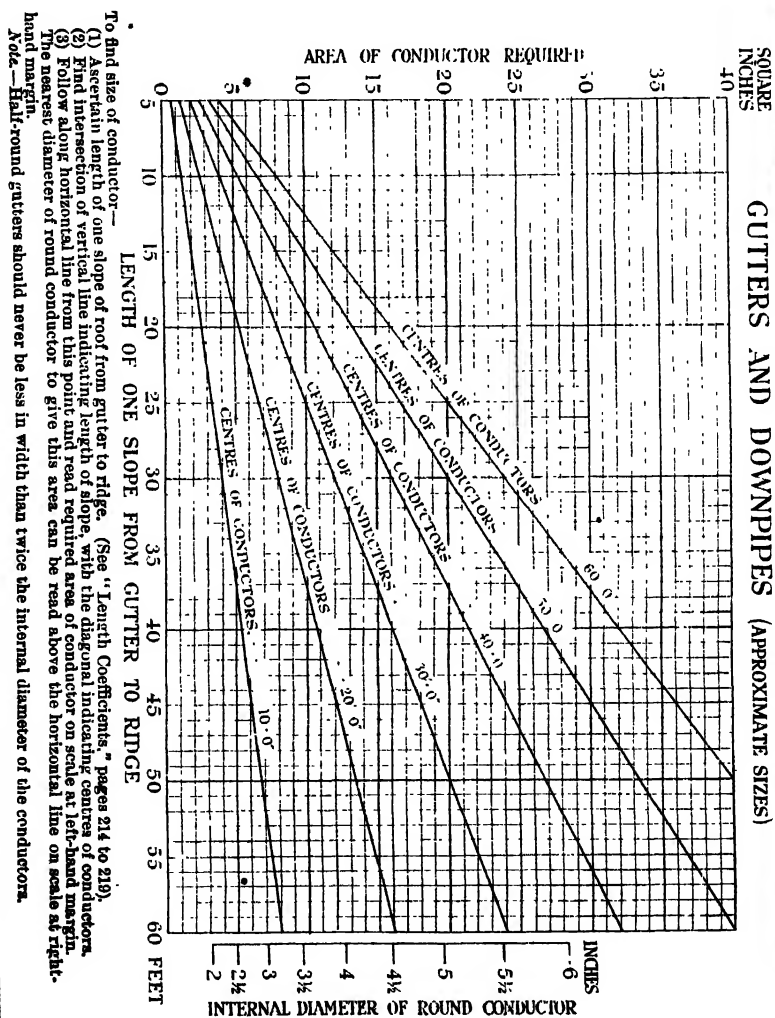
**British Practice.** "Rain-water or downpipes should have a bore or internal area of at least 1 square inch for every 60 super. feet of roof surface in temperate climates. They should be placed not more than 20 feet apart to allow of sufficient fall in the eaves gutters, which should be increased in size if the downpipes are further apart. Eaves gutters should never be less in width than twice the internal diameter of the downpipe; more would be an advantage." (Hurst.)

**American Practice.** "The practice among American architects is to provide about 1 square inch of conductor area for each 75 square feet of roof surface; no conductors less than 2 inches in diameter being used in any case." (Ketchum).

### *American Bridge Company Specifications.*

Span of Roof.	Gutter.	Conductor.
Up to 50 feet.	6 inches.	4 inches every 40 feet.
50 to 70 "	7 "	5 " " 40 "
70 to 100 "	8 "	5 " " 40 "

**Diagram.** The diagram on opposite page is based on the practice of allowing one square inch of downpipe area for each 75 square feet of roof surface. Equivalent areas for rectangular pipes or gutters may be substituted, note being taken of commercial sizes.





**PART IV.**  
**GENERAL.**

**TABULAR CONDITIONS,  
DEFINITIONS,  
LOADS,  
APPLICATIONS  
OF THE  
TABLES,  
FOUNDATIONS,  
PROPERTIES,  
WEIGHTS,  
AREAS,  
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## PART IV.—GENERAL

The utility of this handbook depends largely on the extent to which by a proper use of the tables tedious calculations from first principles are eliminated and the problem of design is reduced to the comparatively simple operation of making an appropriate selection from the number of suitable sections available.

The various notes have been written to explain as fully and as clearly as possible not only the direct applications of the tabular values, but also the methods by which these can be adapted to suit variations of stress, load, support, deflection, and other conditions ordinarily met with in practice.

Parts I., II., III., and V. are each intended to treat of one subject only, the notes to these Parts being confined as strictly as possible to matter directly applicable to the particular tables.

This Part, on the contrary, as the sub-title of "General" indicates, includes all matter applicable to variations of the tabular conditions in addition to useful data, general formulæ, and mathematical tables purposely omitted from the other parts of the book.

The contents of the book generally are intended to apply to structural steelwork in buildings and all forms of construction of a similar nature on which the principal loads are static.

Notwithstanding this distinction certain of the notes and formulæ are of general application, as are the tabulated properties, with the exception of the "Maximum moments of resistance in foot tons," pages 60-67, Part I., which are based on an extreme fibre stress of 7.5 tons per square inch.

### Attention is directed to the following features:—

The arrangement of the overall dimensions, safe loads, composition, weight per foot, and properties of each simple and compound section on the corresponding lines of two facing pages.

The indication of web buckling, deflection, and rivet pitch limitations by the free use of zigzag lines and italics.

The tables of minimum spans in feet for various rivet pitches.

The tables of compound girders arranged in descending order of carrying capacity.

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**The tables of safe loads on steel joists embedded in concrete.**

**The treatment of stanchions by the Moncrieff formulæ for the practical condition of both ends flat.**

**The stanchion eccentricity coefficients relative to the axes of least and greatest radii of gyration, and the treatment of eccentric loading by means of these.**

**The alignment charts for the rapid approximation of the safe loads, &c., on stanchions, and of the intensities of normal wind pressures.**

**The material of each section is structural mild steel having an ultimate tensile strength of 28 to 33 tons per square inch, in accordance with the specification of the Engineering Standards Committee.**

**The tabulated loads are based on the undernoted conditions.**

### **PART I.—BEAMS.**

- (a) Static loading uniformly distributed over the entire length of the effective span.
- (b) The inclusion of the weight of the beam in the load.
- (c) Each end of the beam being simply supported, not fixed.
- (d) The laterally unsupported length of the compression flange not exceeding 30 times its breadth.

#### **Safe loads. Additional condition.**

- (e) A working tensile or compressive stress of 7.5 tons per square inch at the extreme fibres, corresponding to a factor of safety of four.

### **PART II.—STANCHIONS.**

- (f) Static concentric loading.
- (g) Each end of the stanchion being flat.
- (h) The average working compressive stress per square inch for each ratio of slenderness as determined by the Moncrieff formulæ.

**The properties of the simple sections (those of certain angles and tees excepted) have been taken, by permission, from the lists of the Engineering Standards Committee.**

**These values are for the exact profiles of the British Standard dimensions, accurate allowances having been made for rounded corners, fillets, and tapered flanges.**

**The angles and tees excepted above are of thicknesses for which properties are not given by the Committee, and these, in addition to the properties of all compound sections, have been calculated by the technical department of the Company.**

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### DEFINITIONS AND FUNDAMENTAL PRINCIPLES OF STATICS.

*Through indiscriminate use in the past, certain terms have acquired ambiguity of meaning.*

*Present practice tends to eliminate such ambiguity by attaching to each term a particular meaning, as in the following list of definitions, which has been compiled from the leading authorities on the subject.*

**A Static Load** is a stationary load producing no variation of stress intensity, or which is increased gradually from zero up to its maximum amount.

**Reactions** are the pressures at the points of support, due to the loads. The sum of the reactions is invariably equal to the sum of the loads. For values of reaction, see pages 258-263.

**The External Forces** which act on a structure are the loads (dead and live) and the reactions due to these.

**Stress** is the mutual action at the interface between two adjacent portions of a body subjected to the action of external forces.

**Tensile Stress or Tension** is the stress due to the action of two external forces tending to pull the molecules of a body apart.

**Compressive Stress or Compression** is the stress due to the action of two external forces tending to push the molecules of a body together.

**Shearing Stress or Shear** is the stress due to the action of two equal and opposite parallel external forces, tending to make the molecules at two adjoining planes of a body slide past one another.

**Stresses and Forces** are measured in tons or pounds.

**Unit Stress** is a total stress of one ton or one pound.

**Unital Stress** or stress per unit of area or intensity of stress is the quotient obtained by dividing the total stress developed uniformly over a cross section by the area of the cross section. Unital stress is expressed in tons or lbs. per square foot or per square inch.

**Ultimate Strength or Ultimate Stress** are interchangeable terms, meaning the maximum unital stress which can be developed in the material before rupture takes place.

**Working Stress and Factor of Safety.** See page 253.

**Stress and Strain.** Confiction of meaning has arisen through the frequent use of the terms "stress" and "strain" as if they were synonymous. In present practice, "strain" is held to mean the effect of a "stress," and to avoid confusion, it is suggested that the term "deformation" should be substituted.

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### DEFINITIONS, ETC.—(continued).

**Deformation** is the amount of the resulting change in bulk or shape of a body subjected to the action of stress. Deformations are measured by linear units and are usually expressed in inches.

**An Elastic Deformation** is one which disappears entirely on the removal of the external forces causing the stress.

**Permanent Set** is the deformation which wholly or partially remains on the removal of the external forces causing the stress.

**Unital Deformation** or deformation per unit of length is the total deformation or change of length divided by the original length.

**Elastic Limit.** The true elastic limit is the maximum unital stress which can be developed in the material without permanent set resulting.

**Hooke's Law** states that within the true elastic limit stress is directly proportional to the accompanying deformation.

**The Commercial Elastic Limit in Tension** is the maximum unital tensile stress developed in the material up to the moment of marked breakdown of the test piece, viz., at the yield point.

**The Modulus of Elasticity** of the material (Symbol E) is the constant which within the true elastic limit expresses the ratio between unital stress and unital deformation or

$$E = \frac{\text{Unital Stress}}{\text{Unital Deformation}}$$

It may also be defined as that force which would produce in a bar of one unit of cross section, a deformation equal to its original length, provided that Hooke's law were applicable to all stresses without limit.

In present practice the Symbol E denotes **Young's Modulus** or the **Modulus of Longitudinal Elasticity**. The accepted values of E for mild steel are 12,000 to 13,000 tons per square inch, the lower value being generally used for deflection calculations.

**Flexure or Bending** is due to the simultaneous action of tension, compression, and shear. It occurs when an external force or combination of forces applied to a member causes the originally straight axis of the member to assume the form of a curve.

**Beam** is a generic term in statics, applied to a structural member subject to flexure. Beams are ordinarily horizontal members, supporting loads acting vertically.

Members may be subjected to flexure combined with tension or with compression. The main tie of a roof used to support a ceiling, or for lifting loads from a floor is a familiar case of flexure and tension, while rafters supporting intermediate purlins, and stanchions eccentrically loaded or subjected to wind pressure are in a state of flexure and compression.

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### DEFINITIONS, ETC.—(continued).

**A Simple Beam** is a horizontal member simply supported at the ends, so that all parts have free movement in a vertical plane under the influence of vertical loads.

**Distribution of Stress** in a flexed or bent beam. On the convex side the fibres are elongated by tension, and on the concave side the fibres are shortened by compression. Simultaneously, shear is taking place between each vertical plane of the member and the one adjoining. As the stresses change in kind, from tension to compression, it follows that at the surface in the depth of the beam where the change in kind of stress takes place, the intensity of stress is zero.

**The Neutral Axis** is the name given to this surface of zero stress. For a section of mild steel it passes through the centre of gravity or centre of area, as the ultimate tensile and compressive strengths of this material are taken as equal in value. The neutral axis of a symmetrical mild steel section is at the middle of its depth.

**Extreme Fibres** are the fibres of infinitesimal thickness at the surface or edge of the section most remote from the neutral axis, the distance being measured in a direction perpendicular to the neutral axis.

**Extreme Fibre Stress.** It can be shown that within the elastic limit, the deformation, and consequently the stress in any fibre is proportional to its distance from the neutral axis, and that the intensity of stress increases as the distance increases. The maximum intensity of stress is reached at the extreme fibres, and therefore the maximum permissible unital stress at the extreme fibres is also the tensile or compressive working stress.

**A Stanchion or Strut** is a structural member, conventionally vertical and of a height not less than 8 to 10 times its least lateral dimension.

Vertical loading on such a member produces direct or axial compression, accompanied by the development of flexural stresses due to the tendency of the stanchion to fail by buckling or bending in a lateral direction.

The maximum intensity of the compressive stress due to flexure occurs at the extreme fibres of the section on the concave side.

The working stress for a stanchion is therefore the maximum permissible unital stress at the extreme fibres, and is a fraction of the sum of the maximum axial and flexural compressive stresses developed at the point of failure, as determined by a suitable stanchion formula.

**Positive and Negative Forces.** Retaining the convention that a beam is a horizontal member, loads or external forces acting downwards are taken as negative, and reactions or external forces acting upwards are taken as positive.

**The Moment of a Force** about any point is the value of the force multiplied by its leverage or distance from the point, measured in a direction perpendicular to the line of action of the force. A moment being a compound quantity is expressed in foot tons or inch tons.

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### DEFINITIONS, ETC.—(continued).

The Laws of Equilibrium assert that for a system of vertical forces acting in one plane—

- (a) the algebraic sum of all the vertical forces must equal zero.
- (b) the algebraic sum of the moments of all the vertical forces must equal zero.

In other words, as the algebraic sum is the difference of positive and negative values, it is apparent that the negative, or downward forces, and the positive, or upward forces, and their respective moments must be equal and opposite to each other.

**End Reactions.** By the first of the laws of equilibrium the sum of the reactions must equal the sum of the loads.

In a simple beam, each end reaction will equal  $\frac{1}{2}$  the sum of the loads for the conditions of:—

- (a) Loading uniformly distributed.
- (b) A single load concentrated at the centre of the span.
- (c) Any system of concentrated loads in pairs of equal value or partially distributed loads of equal value disposed symmetrically with reference to the points of support.

By the second of the laws of equilibrium the sum of the moments of the reactions must equal the sum of the moments of the loads.

This is the general statement from which the values of the reactions at each support for any symmetrical or unsymmetrical system of loading on a simple beam may be ascertained.

See pages 258-263 for formulæ and numerical example.

### SHEARING FORCE AND BENDING MOMENT.

*NOTE.—As a matter of convenience, in the immediately following paragraphs relating to shear and bending moment, the left hand support is considered to be the point of origin.*

**Vertical Shear** is the measure of the shearing tendency which occurs at every imaginary transverse section in a beam. The vertical shear at any such transverse section is equal to the algebraic sum of all the external forces to the left of the section, and is expressed in tons or lbs.

Vertical shears are termed positive or negative according to the relative values of the loads and reactions to the left of the imaginary transverse section.

The **Maximum Positive Vertical Shear** in a simple beam occurs over the left hand support and is equal in amount to the left hand reaction. Similarly the maximum negative vertical shear occurs over the right hand support and is equal in amount to the right hand reaction.

**Zero Point.** It follows that the vertical shear must pass through zero at the intermediate point where the sign of shear changes from positive to negative.

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### DEFINITIONS, ETC.—(continued).

**Location of Zero Point.** This point may be located for any system of static loading by the following method :—

Beginning at the left hand support, add the successive values of the loads to the right until their sum exceeds the value of the left hand reaction. The point at which the vertical shear passes through zero occurs immediately under the load causing the excess.

If the loading is uniformly distributed or symmetrically disposed, relative to the supports, the point of zero shear is at the centre of the span.

See pages 258-263 for formulæ and diagrams.

**The Bending Moment** at any imaginary transverse section of a beam is the measure of the action of the external forces tending to cause rotation about the section. The bending moment at any such section is equal to the algebraic sum of the moments of all the external forces to the left of the section. It is expressed in inch tons or foot tons. Moments to the left of the section acting upwards are taken as positive, and those to the left of the section acting downwards are taken as negative.

**The Maximum Bending Moment** in a simple beam supporting any system of static loading, occurs at the point at which the vertical shear passes through zero.

If the loading is uniformly distributed or symmetrically disposed relative to the supports, the maximum bending moment occurs at the centre of the span.

In a cantilever the maximum bending moment occurs at the supports irrespective of the position of the loads.

**Minimum Bending Moment.** In a simple beam the bending moment at the supports is zero.

See pages 258-263 for formulæ and diagrams.

**Deflection** is the measure of the vertical displacement of any point of a loaded beam from its position when the beam is unloaded. Deflection is expressed in inches.

In steel structural work for buildings it is the practice to use formulæ for the calculation of the maximum deflection due to the flexural stresses only. Deflection due to shear, which amounts to about 3% of the total for ordinary structural sections is neglected.

**The Maximum Deflection** in the case of a simply supported beam may be taken as at the point of maximum bending moment.

**The Investigation of the Strength of Beams** is governed by the three following laws :—

- (a) The sum of all tensile stresses must equal the sum of all compressive stresses.
- (b) The resisting shear must equal the vertical shear.
- (c) The moment of resistance must equal the bending moment.

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### DEFINITIONS, ETC.—(continued).

or summarized,

- (d) In order that equilibrium may obtain, the external forces acting at any imaginary transverse section of a beam must be equalled by the internal resisting forces, and their moments must also be equal.

**The Moment of Resistance** at any imaginary transverse section of a beam is the measure of the action of the internal forces which resist the rotary tendency caused by the external forces. The moment of resistance at any such section is equal to the sum of the moments of all the tensile and compressive stresses in the material at the section, acting as a couple. It is usually expressed in inch-tons, but if equated to the bending moment in foot-tons, it must be expressed in the same terms as the latter. The moment of resistance of a section is most conveniently derived from the corresponding tabular value of moment of inertia, as the direct determination involves the integral calculus.

The maximum moment of resistance may be taken as the criterion of the strength of a beam to resist flexure.

**Properties** of a section are values dependent upon its profile or shape only, and which form a basis for the determination of its strength. All the tabulated properties are calculated with reference to central axes.

**A Central Axis** is one which passes through the centre of gravity or, more correctly, through the centre of area of the figure or profile of the section.

**Central Axis and Neutral Axis** coincide for a section of mild steel, this material being taken as of equal strength in tension and in compression. The former term is generally used in connection with values such as properties involving areas or linear dimensions only, the latter term is used in connection with values such as moment of resistance involving stress.

**A Principal Axis** is a central axis with reference to which the moment of inertia is maximum or minimum.

**An Axis of Symmetry** is a principal central axis dividing the profile of a section into two portions of equal area and shape.

**An Asymmetrical Axis** is a central axis which does not divide the profile symmetrically. It may or may not be a principal axis.

**The Ellipse of Inertia** is an ellipse constructed to show the relations between the moments of inertia and radii of gyration for different central axes.

**Moment of Inertia** is the basis property for taking account of the fact that the moment of stress varies as the square of the distance from the neutral axis. If it is supposed that a transverse section of a beam is divided into elementary areas, then the moment of inertia is equal to the

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### DEFINITIONS, ETC.—(continued).

sum of the products obtained by multiplying each elementary area by the square of its perpendicular distance from the central axis. It is expressed in inches.<sup>4</sup>

**Modulus of Section** at any imaginary transverse section of a beam is the measure of the resisting moment of the beam at the section. It is expressed in inches.<sup>5</sup>

The maximum modulus of section is largely used as a direct basis of comparison of the strength of a beam to resist flexure, being equal to the moment of resistance for an extreme fibre stress of one ton per square inch.

Modulus of Section is sometimes termed Moment of Resistance, but this is apt to lead to confusion when practical working stresses have to be taken into account.

**Radius of Gyration** is generally used as the basis of comparison of the strength of a stanchion to resist buckling or bending. It is a linear dimension expressed in inches.

It is equal to the perpendicular distance from the central axis to such point as, if all the area were there concentrated, the moment of inertia would be the same.

*The terms used in this book have the foregoing definite meanings unless where specifically qualified by the context.*

### NOTATION.

The same notation is used throughout the book.

The same symbol is used on occasion to denote different quantities when these cannot possibly be mistaken for each other.

Subscripts denote values relative to particular axes, or particular applications of a general symbol.

For convenience of reference, the less familiar symbols and their meanings are repeated where necessary.

### ECONOMICAL CONSIDERATIONS AFFECTING DESIGN.

The general problem of steelwork design is to provide in a structure a sufficient area of material distributed in such a manner that the external forces will be safely resisted by the internal forces.

The correct theoretical solution will necessarily be qualified by the important practical considerations of the location of the principal members, available commercial sizes, cost of manufacture, and time for completion.

It should be borne in mind that sections of the exact areas required are not always procurable, that some forms of material are cheaper and more readily obtainable than others, and that material is cheaper than workmanship.

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### ECONOMICAL CONSIDERATIONS—(continued).

It follows that the design in which the area of each member is reduced to the lowest possible limits consistent with safety, may be the most correct theoretically, but it may also be very costly to produce.

On the other hand, the best design is that which accomplishes the object in view in the most economical fashion, and which may be executed in the shortest period of time.

These desiderata are attained by aiming at simplicity of workmanship throughout, by avoiding complicated forms of lattice work and connections, and by using only such sections as are readily obtainable, preferably those which are always in stock. See page 6.

The number and positions of the stanchions, beams, and roof trusses forming a steel structure must depend upon the scheme of architecture and the purpose for which the building is intended.

No definite rules can be given, but where no restrictions are imposed the following may be noted as tending to economy in the use of steelwork.

Manufacturing and erection costs are kept low by the adoption of a convenient unit such as 15 feet by 15 feet for the centres of stanchions and spans of main beams, and adhering throughout to the unit arrangement decided upon.

By this means stanchions, beams, and connections may be standardised and the minimum number of drawings and templates are required.

### BEAMS.

For a given load beams of short span are relatively more economical per unit of length than beams of long span.

This will generally hold good even taking the necessary supports into account, especially if the latter are steel stanchions, except in the case of a moving load which may act at its full value on each support irrespective of spacing.

For usual conditions of loading, the most economical beam is the deepest available steel joist or compound girder of the required strength. It may even be considerably in excess of the required strength and remain less costly than a shallower section. See Part I., page 109.

For this reason, under certain circumstances, it may be cheaper to increase the overall height of a floor or of a building than to restrict the beam depth.

This applies particularly to large steel framed buildings of the warehouse class having thin external walls and few, if any, important internal partitions.

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### ECONOMICAL CONSIDERATIONS—(continued).

**Beams of unsymmetrical section** are not relatively economical.

The dotted and full zigzag lines and the italics of the beam tables, Part I., may be said to mark the economical limits of the various sections and types.

By choosing sections within those limits the additional costs of stiffeners and special rivet pitches are avoided and undue deflection prevented.

*Special deflection considerations are treated later.*

### STANCHIONS.

The full zigzag lines of the stanchion tables, Part II., mark the economical limits of height. To the right of these lines the loads for each increase of height decrease more rapidly than do these to the left.

For a series of superimposed stanchions an economical method is to select a simple joist section for the topmost storey, and retaining the same joist section to the foundation, add the increase of area required at each succeeding lower floor level by means of plates riveted to each flange.

Eccentric loading is costly and should be avoided if possible.

### LOADS.

The calculation of the value and condition of the load to be supported is in every case a preliminary necessary to the design or selection of a structural member.

This is a matter of great importance, as the efficiency and economy of a structure must depend to a very large extent on the degree of approximation of the assumed load values, to those actually realised in practice.

Consider a building of steel skeleton construction.

The necessary load calculations are accomplished most conveniently in the reverse order of the building operations.

Commence at the roof or highest portion of the structure, and work down through each floor in succession.

At each level take the secondary members such as flooring beams before the main girders, and main girders before stanchions.

Note the total load value transmitted to each stanchion, at each tier, and finally arrive at the total load on each foundation.

### CONDITIONS OF LOADING.

On beams the loading may be—

- (a) Uniformly distributed over entire length of effective span.
- (b) Concentrated at one or more points.
- (c) Unequally distributed.
- (d) Any combination of a, b, and c.

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## LOADS—(continued).

On stanchions the loading may be—

- (e) Concentric.
- (f) Balanced.
- (g) Eccentric. •
- (h) Any combination of e, f, and g.

**A structure is designed to support both dead and alive loads.**

The total dead load comprises the weight of the structure itself, and all permanent loads, i.e., roof coverings, floors, partitions, walls, and heavy fixtures.

The total live load comprises the weight of all variable or moveable loads, i.e., wind pressure, snow, water in tanks, people, furniture, goods, or merchandise.

Machinery, overhead travelling cranes, &c., require special consideration.

Dead and live loads should be calculated in accordance with the requirements of the local Building Authority.

In the absence of specific regulations, the following rules based on the London County Council (General Powers) Act, 1909, may be followed.

- (1) The dead load shall consist of the actual weight of walls, floors, roofs, partitions, and all other permanent construction.
- (2) The live or superimposed load shall be estimated as equivalent to the following dead load.

## FLOORS.

Description of Building.	Load in lbs. per square foot of floor area.
Human habitation or domestic building, -	70
Office, counting-house or similar building,	100
Workshop or retail shop, - - - -	112
Building of the warehouse class, - -	Not less than 224.

(Table continued overleaf).

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LOADS—(continued).

## ROOFS.

Description.	Load in lbs. per square foot of roof area including wind pressure.
Angle of inclination to the horizontal greater than 20 degrees, - . . . . .	28 Measured on slope.
All other roofs, - . . . . .	56 Measured horizontally.

(3) If the superimposed load is to exceed that specified for its class, the excess shall be provided for.

(4) All buildings shall be designed to resist safely a horizontal wind pressure of not less than 30 lbs. per square foot of the upper two-thirds of the exposed surface.

## BEAMS SUPPORTING BRICK WALLS.

When a beam or girder is used to support a brick wall over an opening, the value of the dead load may vary according to several conditions.

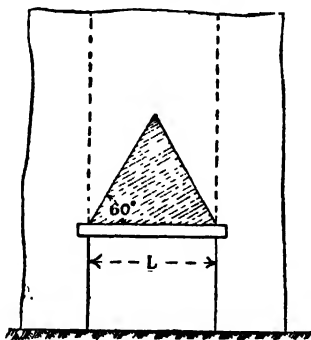


FIG. 1.

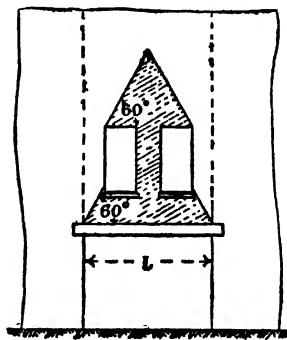


FIG. 2

The usual practice in this country is to take a load uniformly distributed equivalent to the weight of the brickwork enclosed by the equilateral triangle shown in Fig. 1.

If there are windows or other openings in the wall, the usual course is to take the weight of the area of brickwork shown shaded in Fig. 2.

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## BEAMS SUPPORTING BRICK WALLS—(continued).

The load should be taken as equivalent to the whole mass of brickwork enclosed by dotted lines in Figs. 1 and 2 if:—

- (a) The brickwork is not thoroughly bonded throughout.
- (b) The breadth of each abutment is less than half the span.
- (c) Great rigidity is required.

In buildings of steel skeleton construction, the entire weight of the brickwork, terra cotta or other material within the rectangle formed by two stanchions and two beams at succeeding floor levels should be taken as the load on the lower beam.

In every case add the load due to any portion of roof or floor supported.

For permitted ratios of deflection for beams supporting brick walls, see page 271.

## WEIGHTS OF MATERIALS.

The following table of average weights of materials has been compiled from various authoritative sources:—

### APPROXIMATE WEIGHTS OF MASONRY, TIMBER, METALS, &C.

LS. PER CUBIC FOOT.

#### MASONRY.

Asphalte, . . . . .	140—150	Concrete, Reinforced, . . . . .	150—160
Brickwork pressed, . . . . .	130—150	Granite, . . . . .	140—190
" ordinary, . . . . .	110—130	Limestone, Ashlar, . . . . .	140—170
" soft, . . . . .	90—100	" , Rubble, . . . . .	130—150
Cement (Portland), . . . . .	86—94	Sandstone, . . . . .	130—150
Concrete, . . . . .	110—140	Slate, . . . . .	160—180

#### TIMBER.

Elm, . . . . .	34—36	Oak (American), . . . . .	48—54
Greenheart, . . . . .	60—70	Pitch Pine, . . . . .	42—48
Jarrah (Wood Paving), . . . . .	60—63	Red Pine and Spruce Fir, . . . . .	30—44
Larch, . . . . .	31—35	Teak, . . . . .	41—55
Oak (English), . . . . .	48—60	Yellow Pine (American), . . . . .	30—32

#### METALS AND ALLOYS.

Aluminium, . . . . .	160—167	Iron, Wrought, . . . . .	485
Brass, . . . . .	525—530	Lead, . . . . .	710
Copper, Sheet, . . . . .	548	Steel, Cast, . . . . .	492
" , Wire, . . . . .	555	" , Rolled, . . . . .	490
Gunmetal, . . . . .	528	White Metal, . . . . .	456
Iron, Cast, . . . . .	450	Zinc, . . . . .	437

(Table continued overleaf.)

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## WEIGHTS OF MATERIALS—(continued)

*The following table of average weights of materials has been compiled from various authoritative sources:—*

### APPROXIMATE WEIGHTS OF MISCELLANEOUS MERCHANDISE LBS. PER CUBIC FOOT.

Barley, - - - - -	35—40	Hay and Straw in Bales, -	14—19
Cement in Bags, - - -	84	Leather in Bales, - - -	16—23
Cement in Barrels, - -	62—82	Lime in Barrels, - - -	50—60
Coal (Broken), - - -	80—95	Ons, - - - - -	25—30
Coke, - - - - -	40—50	Paper, - - - - -	10—64
Corn, - - - - -	30—35	Plaster in Barrels, - -	50—60
Cotton in Bales, - - -	12—43	Rags in Bales, - - -	7—36
Cotton Goods, - - -	11—37	Rope, - - - - -	40—45
Crockery in Crates, -	35—40	Sugar, - - - - -	45—50
Flour, - - - - -	40—45	Wheat, - - - - -	40—45
Glass, - - - - -	160—190	Wool in Bales, - - -	5—28
Glass in Boxes, - - -	60	Woollen Goods, - - -	13—22

1 cub. ft. of Fresh Water = 62.5 lbs.

1 gallon " " = 10 "

36 cub. ft. " " = 1 ton.

224 gallons " " = 1 ton.

Weight of Sea Water = 1.026 x Weight of Fresh Water.

## SELECTION OF SECTIONS.

Assuming that the value and condition of a load is known, the question of selecting a suitable section from the tables may now be considered.

### For the tabular conditions of stress load and support.

#### BEAMS.—PART I.

A section is suitable as regards flexure, deflection, web buckling, and rivet pitch, provided the tabular safe load for the required span is:—

- (a) Not less than the actual load.
- (b) Not printed in italics.
- (c) Not to the left of a dotted zigzag line.
- (d) Not to the right of a full zigzag line.

#### STANCHIONS.—PART II.

A section is suitable as regards strength provided that the tabular load for the required height is:—

- (a) Not less than the actual concentric load.
- (b) Not less than the equivalent concentric load value of an actual load eccentric about the axis of least radius of gyration.

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### FACTOR OF SAFETY AND WORKING STRESS.

The **factor of safety** is the number by which the ultimate strength of the material must be divided to give the working stress.

The **working stress** or safe stress is the highest permissible fraction of the ultimate strength determined by practice to give a proper degree of security against the rupture of any portion of the material.

The **factor of safety** and **working stress** ensure that the maximum stresses developed in any member must never approach the ultimate strength of the material by making reasonable provision for :—

- (a) Undiscoverable and unavoidable imperfections of material and workmanship.
- (b) Deterioration of material due to fatigue or oxidation.
- (c) The possibility under unforeseen circumstances of an increase of the amount or change in the nature of the load calculated to be supported.

Usual factors of safety and corresponding working stresses in tons per square inch are :—

Condition of Loading.	Factor of Safety.	Working Stresses.	
		Tension or Compression.	Shear.
For Stationary Loads, - - -	4	7·5	5·5 to 6
For Moving Loads not applied with impact (see also pages 295-296), -	6	5	3·6 to 4
For Temporary Work, - - -	3	10	7·3 to 8

### BEAMS. PART I.

#### Variations of the Tabular Conditions.

#### Formulae for Equivalent Tabular Loads.

By means of the following formulæ, equivalent tabular loads may be ascertained for the variations of stress, factor of safety, load and support conditions, ordinarily met with in practice.

As the included weight of the beam is uniformly distributed, the results obtained for concentrated loading are not strictly accurate, but are sufficiently so for practical purposes.

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VARIATIONS—(continued).

## FORMULÆ FOR EQUIVALENT TABULAR LOADS.

### SAFE LOAD TABLES.

Description.	Support Condition.	Load Condition.	Equivalent Tabular Load for		
			Extreme Fibre Stress.	Factor of Safety.	
			Modified	Tabular	
Beam.	Both ends simply supported.	Loading uniformly distributed.	$\frac{7.5 W}{f}$	$W$	$\frac{W F}{4}$
"	"	Single load at centre of span.	$\frac{15 W}{f}$	$2 W$	$\frac{W F}{2}$
"	Both ends fixed or encastré.	Loading uniformly distributed.	$\frac{5 W}{f}$	$\frac{W}{1.5}$	$\frac{W F}{6}$
"	"	Single load at centre of span.	$\frac{7.5 W}{f}$	$W$	$\frac{W F}{4}$
Cantilever.	One end fixed or encastré.	Loading uniformly distributed.	$\frac{30 W}{f}$	$4 W$	$W F$
"	"	Single load at extreme outer end.	$\frac{60 W}{f}$	$8 W$	$2 W F$

W = actual load in tons.  $f$  = extreme fibre stress in tons per square inch.  
F = factor of safety.

When the tabular conditions are modified, deflection, web-buckling, and rivet pitch limitations should be noted.

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VARIAIONS—(continued).

## FORMULÆ FOR EQUIVALENT TABULAR LOADS. BREAKING LOAD TABLES.

Description.	Support Condition.	Load Condition.	Equivalent Tabular Load for		
			Extreme Fibre Stress.		Factor of Safety.
			Modified.	Tabular Breaking.	Modified.
Beam.	Both ends simply supported.	Loading uniformly distributed.	$\frac{30 W}{f}$	W	W F
"	"	Single load at centre of span.	$\frac{60 W}{f}$	2 W	2 W F
"	Both ends fixed or encastré.	Loading uniformly distributed.	$\frac{20 W}{f}$	$\frac{W}{1.5}$	$\frac{W F}{1.5}$
"	"	Single load at centre of span.	$\frac{30 W}{f}$	W	W F
Cantilever.	One end fixed or encastré.	Loading uniformly distributed.	$\frac{120 W}{f}$	4 W	4 W F
"	"	Single load at extreme outer end.	$\frac{240 W}{f}$	8 W	8 W F

W = actual load in tons.  $f$  = extreme fibre stress in tons per square inch.  
F = factor of safety.

When the tabular conditions are modified, deflection, web-buckling, and rivet pitch limitations should be noted.

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**VARIATIONS—(continued).**

### FORMULÆ FOR EQUIVALENT TABULAR LOADS.

*Examples:—*

(a) Required a suitable section for a load of 14·08 tons (including weight of beam) distributed uniformly over a span of 12 feet, the extreme fibre stress not to exceed 6 tons per square inch, corresponding to a factor of safety of 5.

The equivalent tabular load  $W_r$  to be referred to is:

$$\left. \begin{aligned} W_r &= \frac{7.5W}{f} \\ &= \frac{7.5 \times 14.08}{6} \\ &= 17.6 \text{ tons.} \end{aligned} \right\} \text{ or } \left\{ \begin{aligned} W_r &= \frac{WF}{4} \\ &= \frac{14.08 \times 5}{4} \\ &= 17.6 \text{ tons.} \end{aligned} \right.$$

On referring to 17.6 tons in the table on page 16, Part I., it is found that a steel joist 10" × 6" × 42 lbs. is suitable.

(b) Required a suitable section of tee as a cantilever for a load of  $\frac{1}{2}$  ton concentrated at a point 5 feet from the support, the extreme fibre stress not to exceed 10 tons per square inch corresponding to a factor of safety of 3.

The equivalent tabular load  $W_r$  to be referred to is:—

$$\left. \begin{aligned} W_r &= \frac{240W}{f} \\ &= \frac{240 \times .5}{10} \\ &= 12 \text{ tons.} \end{aligned} \right\} \text{ or } \left\{ \begin{aligned} W_r &= 8 WF \\ &= 8 \times .5 \times 3 \\ &= 12 \text{ tons.} \end{aligned} \right.$$

On referring to page 92, Part I., it is found that the tabular loads for  $4" \times 5" \times \frac{1}{2}"$  tee for 5 feet span is 11.9 tons, therefore this section is suitable.

### LATERAL SUPPORT.

Experience has shown that the conventional estimate of the strength of a beam without lateral support is somewhat low. This is specially applicable to a steel joist or compound girder, as the solid web assists the tension flange in sustaining the compression flange, and the buckling which would otherwise take place is prevented to a large extent.

It has been proved by experiment that when the laterally unsupported length of a beam becomes 80 times the flange breadth, the normal strength is reduced by about one-third.

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### LATERAL SUPPORT—(continued).

Should the laterally unsupported length of the compression flange of a beam exceed 30 times its breadth it is recommended by certain authorities that the loading be reduced in the following proportions :—

Distance between Lateral Supports.	Safe Load Uniformly Distributed.
40 times flange width,	$\frac{7}{8}$ tabular load.
50   "   "   "	$\frac{8}{10}$ "   "
60   "   "   "	$\frac{9}{10}$ "   "
70   "   "   "	$\frac{10}{10}$ "   "

These proportions apply also to the equivalent tabular loads found by formulae, pages 254-255.

In structural steelwork for buildings the expert engineer is permitted a considerable latitude in interpreting the above recommendation.

### BENDING MOMENT AND MOMENT OF RESISTANCE.

The law that "for equilibrium the moment of resistance must equal the bending moment," underlies all formulæ for the flexural strength of a beam.

For this reason, when an equivalent tabular load cannot be ascertained immediately by a convenient multiplier (such as 2 for a single concentrated central load), the method adopted by the majority of engineers is to calculate the maximum bending moment due to the system of loading.

Formulae for the calculation of the maximum bending moment or usual systems of loading are given on the following pages.

#### MOMENT OF RESISTANCE TABLES. PART I.

If the maximum bending moment in foot tons is known for any system of loading, a suitable section for the tabular extreme fibre stress of 7.5 tons per square inch or factor of safety of 4, may be selected without further calculation by using the tables of "Compound Girders arranged in Descending Order of Carrying Capacity," Part I, pages 60 to 67 inclusive.

The method of using these tables is given in Part I., page 109.

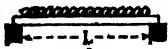
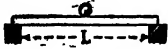
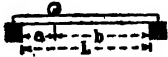
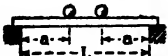
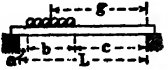
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## LOAD EQUIVALENTS AND MAXIMUM DEFLECTIONS FOR VARIOUS CONDITIONS OF LOADING.

$W_A$  = Superimposed Load.  
 $L$  = Effective Span.

$W_B$  = Total Weight of Beam.  
 $I$  = Moment of Inertia.


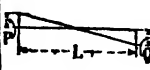

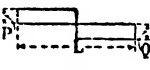
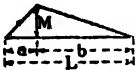
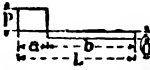
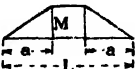
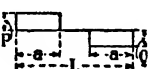

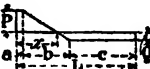
$E$  = Modulus of Elasticity.

Condition of Loading.	Safe Load Factor (approx.).	Equivalent Distributed Load including $W_B$ .	Maximum Deflections.	
			Due to Superimposed Load $W_A$ .	Due to Superimposed Load $W_A$ + Weight of Beam $W_B$ .
 (1)	1	$W_A + W_B$	$\frac{5W_A L^3}{384EI}$	$\frac{5(W_A + W_B)L^3}{384EI}$
 (2)	$\frac{1}{2}$	$2W_A + W_B$	$\frac{W_A L^3}{48EI}$	$\frac{W_A L^3}{48EI} + \frac{5W_B L^3}{384EI}$
 (3)	$\frac{L^2}{8ab}$	$\frac{4ab(2W_A + W_B)}{L^2}$	$\frac{W_A ab(2L - a)}{9KIL} \times \sqrt{\frac{a(2L - a)}{3}}$	
 (4)	$\frac{L}{4a}$	$\frac{4W_A a}{L} + W_B$	$\frac{W_A a (3L^2 - 4a^2)}{48EI}$	$\frac{W_A a (3L^2 - 4a^2)}{48EI} + \frac{5W_B L^3}{384EI}$
 (5)	$\frac{L^3}{(4bg^2 + 8agL)}$ for $g = \frac{b}{2} + c$	For Symmetrical Sections, Working Stress at 7.5 tons per square inch. $E = 12,000$ tons per square inch. $L$ in feet. $\delta$ = Maximum deflection in inches. $K$ = Coefficient, Part I. Then for Case (1) $\delta = KL^2$ and for case (2) $\delta = KL^2 + 1.25$ . Safe Load Factor (approximate). $W$ = Total Load in Tons. $W_T$ = Tabular Load in Tons, Part I. Then $W = W_T \times \text{Safe Load Factor}$ .		

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## BENDING MOMENTS, REACTIONS AND VERTICAL SHEARS FOR VARIOUS CONDITIONS OF LOADING.

$M$  = Maximum Bending Moment.  $L$  = Effective Span.  
 $P$  = Maximum Positive Shear = Left Hand Reaction.  
 $Q$  = Maximum Negative Shear = Right Hand Reaction.

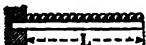
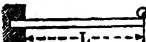
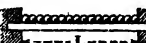
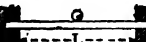
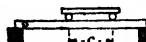
Bending Moments.			Reactions and Vertical Shears.		
Diagram.	Maximum due to		Diagram.	Maximum due to	
	$W_A$ only.	$W_A + W_B$		$W_A$ only.	$W_A + W_B$
 (1a)	$M = \frac{W_A L}{8}$	$M = \frac{(W_A + W_B)L}{8}$	 (1b)	$P = \frac{W_A}{2}$ $Q = \frac{W_A}{2}$	$P = \frac{W_A + W_B}{2}$ $Q = \frac{W_A + W_B}{2}$
 (2a)	$M = \frac{W_A L}{4}$	$M = \frac{W_A L}{4} + \frac{W_B L}{8}$	 (2b)	$P = \frac{W_A}{2}$ $Q = \frac{W_A}{2}$	$P = \frac{W_A + W_B}{2}$ $Q = \frac{W_A + W_B}{2}$
 (3a)	$M = \frac{W_A a b}{L}$	$M = \frac{a(2W_A b + W_B L)}{2L} - \frac{W_B a^2}{2L}$	 (3b)	$P = \frac{W_A b}{L}$ $Q = \frac{W_A a}{L}$	$P = \frac{W_A b}{L} + \frac{W_B}{2}$ $Q = \frac{W_A a}{L} + \frac{W_B}{2}$
 (4a)	$M = \frac{W_A a}{4}$	$M = \frac{W_A a}{2} + \frac{W_B L}{8}$	 (4b)	$P = \frac{W_A}{2}$ $Q = \frac{W_A}{2}$	$P = \frac{W_A + W_B}{2}$ $Q = \frac{W_A + W_B}{2}$
 (5a)	$M = \frac{W_A a c}{L} + \frac{W_B b c^2}{2L^2}$ for $s = a + \frac{b(\frac{b}{2} + c)}{L}$	$M = \frac{W_A a c}{L} + \frac{W_B b c^2}{2L^2}$	 (5b)	$P = \frac{W_A}{L}(\frac{b}{2} + c)$ $Q = \frac{W_A}{L}(a + \frac{b}{2})$	$P = \frac{W_A}{L}(\frac{b}{2} + c) + \frac{W_B}{2}$ $Q = \frac{W_A}{L}(a + \frac{b}{2}) + \frac{W_B}{2}$

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## LOAD EQUIVALENTS AND MAXIMUM DEFLECTIONS FOR VARIOUS CONDITIONS OF LOADING.

$W_A$  = Superimposed Load.  
 $L$  = Effective Span.


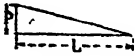
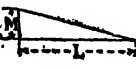
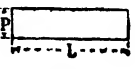
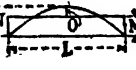
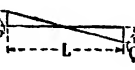
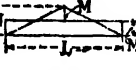
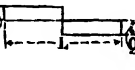

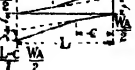
$W_B$  = Total Weight of Beam.  
 $I$  = Moment of Inertia.  
 $E$  = Modulus of Elasticity.

Condition of Loading.	Safe Load Factor (approx.).	Equivalent Distributed Load including $W_B$ .	Maximum Deflections.	
			Due to Superimposed Load $W_A$ .	Due to Superimposed Load $W_A$ + Weight of Beam $W_B$ .
 (6)	$\frac{1}{2}$	$W_A + W_B$	$\frac{W_A L^3}{8EI}$	$\frac{(W_A + W_B)L^3}{8EI}$
 (7)	$\frac{1}{8}$	$2W_A + W_B$	$\frac{W_A L^3}{3EI}$	$\frac{W_A L^3}{8EI} + \frac{W_B L^3}{8EI}$
 (8)	$1\frac{1}{2}$	$W_A + W_B$	$\frac{W_A L^3}{84EI}$	$\frac{(W_A + W_B)L^3}{84EI}$
 (9)	1	$2W_A + W_B$	$\frac{W_A L^3}{192EI}$	$\frac{W_A L^3}{192EI} + \frac{W_B L^3}{84EI}$
 (10)		<p>For Symmetrical Sections, Working Stress at 7.5 tons per square inch. <math>E = 12,000</math> tons per square inch. <math>L</math> in feet. <math>\delta</math> = Maximum deflection in inches. <math>K</math> = Coefficient, Part I. Then for case (6) <math>\delta = KL^2 \times 2.4</math>, and for case (7) <math>\delta = KL^2 \times 8.125</math>.</p> <p>Safe Load Factor (approximate).  <math>W</math> = Total Load in Tons.  <math>W_T</math> = Tubular Load in Tons, Part I.  Then <math>W = W_T \times \text{Safe Load Factor}</math>.</p>		

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## BENDING MOMENTS, REACTIONS, AND VERTICAL SHEARS FOR VARIOUS CONDITIONS OF LOADING.

M = Maximum Bending Moment. I = Effective Span.  
P = Maximum Positive Shear = Left Hand Reaction.  
Q = Maximum Negative Shear = Right Hand Reaction.

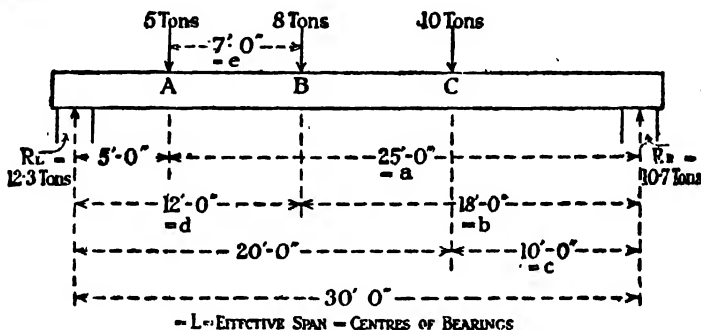
Bending Moments.			Reactions and Vertical Shears.		
Diagram.	Maximum due to		Diagram.	Maximum due to	
	$W_A$ only.	$W_A + W_B$ .		$W_A$ only.	$W_A + W_B$ .
	$M = \frac{W_A L}{2}$	$M = \frac{(W_A + W_B) L}{2}$		$P = W_A$	$P = W_A + W_B$
(6a)			(6b)		
	$M = W_A L$	$M = \frac{(W_A + W_B) L}{2}$		$P = W_A$	$P = W_A + W_B$
(7a)			(7b)		
	$M = \frac{W_A L}{12}$ $N = \frac{W_A L}{8}$ $O = \frac{W_A L}{24}$	$M = \frac{(W_A + W_B) L}{12}$ $N = \frac{(W_A + W_B) L}{8}$ $O = \frac{(W_A + W_B) L}{24}$		$P = \frac{W_A}{2}$ $Q = \frac{W_A}{2}$	$P = \frac{W_A}{2} + \frac{W_B}{2}$ $Q = \frac{W_A}{2} + \frac{W_B}{2}$
(8a)			(8b)		
	$M = \frac{W_A L}{8}$ $N = \frac{W_A L}{4}$	$M = \frac{W_A L}{4} + \frac{W_B L}{8}$ $N = \frac{W_A L}{8} + \frac{W_B L}{12}$		$P = \frac{W_A}{2}$ $Q = \frac{W_A}{2}$	$P = \frac{W_A}{2} + \frac{W_B}{2}$ $Q = \frac{W_A}{2} + \frac{W_B}{2}$
(9a)			(9b)		
	$M = \frac{W_A}{4} (L - c + \frac{c^2}{L})$			$P = \frac{W_A}{4} (1 - \frac{c}{2L})$ $Q = \frac{W_A}{4} (1 - \frac{c}{2L})$	
(10a)			(10b)		

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## REACTIONS, BENDING MOMENTS, Etc.

The following example shows the method of calculating the reactions, vertical shears, maximum bending moment, and deflection, due to a non-uniform system of loading on a simply supported beam.

The selection of a suitable section from the tables of Part I. is indicated also.



- L - EFFECTIVE SPAN - CENTRES OF BEARINGS

$$\text{Total load} = W = A + B + C = 5 + 8 + 10 = 23 \text{ tons}$$

$$\begin{aligned} \text{Left hand reaction} = R_L &= \frac{(A \times a) + (B \times b) + (C \times c)}{L} \\ &= \frac{5 \times 25 + 8 \times 18 + 10 \times 10}{30} = 12.3 \text{ tons.} \end{aligned}$$

$$\begin{aligned} \text{Right hand reaction} = R_R &= W - R_L = 23 - 12.3 \\ &= 10.7 \text{ tons} \end{aligned}$$

$R_L$  and  $R_R$  are also respectively equal to the maximum positive and maximum negative vertical shears.

Position of zero shear and maximum bending moment.  $M_r$  = maximum bending moment in ft. tons.

Add the loads in succession from left hand. Then B added to A exceeds  $R_L$ .  $\therefore$  Zero shear and maximum bending moment occur at B.

Value of maximum bending moment.

$$M_r = R_L \times d - A \times e = 12.3 \times 12 - 5 \times 7 = 112.6 \text{ ft. tons.}$$

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### SELECTION OF A SUITABLE SECTION.

#### (a) Method 1, by moment of resistance.

The moment of resistance in foot tons required is equal to the maximum bending moment = 112.6 ft. tons.

Refer to Part I., page 65.

The girder composed of 1 steel joist 16" x 6" and 2 plates 10" x  $\frac{3}{4}$ " weighing 115½ lbs. per foot has a maximum moment of resistance of 115.4 foot tons, therefore this section is suitable as regards flexure.

Refer to Part I., page 22.

As the safe load is printed in ordinary type and is between the dotted and full zig-zag lines, the section is suitable also as regards rivet pitch, web buckling and deflection for ordinary conditions.

#### (b) Method 2, by modulus of section. Symbol Z.

The modulus of section Z required is equal to the maximum bending moment in foot tons  $\times 1.6$  or  $Z = M_r \times 1.6 = 112.6 \times 1.6 = 180.16$  inches.<sup>3</sup>

Refer to Part I., page 22.

The section referred to above has a maximum modulus of section of 184.6 inches<sup>3</sup>.

#### (c) Method 3, by equivalent distributed tabular load. $W_x$ .

The equivalent distributed tabular load to be referred to is equal to 8 times the maximum bending moment in foot tons divided by the effective span in feet or

$$W_x = \frac{8M_r}{L} = \frac{8 \times 112.6}{30} = 30 \text{ tons approx.}$$

Refer to Part I., page 22.

The tabular load for 30 feet span for the section referred to above is 30.8 tons.

If the depth is unrestricted, reference may be made to Part I., page 16.

Section steel joist, B.S.B., 30, 24" x 7½", weighing 100 lbs. per foot, has a maximum moment of resistance in foot tons = 138.2, a maximum modulus of section = 221.2 inches<sup>3</sup>, and the tabular load for 30 feet span = 36.8 tons. This is the lightest suitable section available.

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## MOMENT OF RESISTANCE—(continued).

By the following formulæ, the equivalent tabular moment of resistance in foot tons corresponding to any specified extreme fibre stress or factor of safety may be ascertained from the known value of the maximum bending moment in foot tons.

$M_r$  = Maximum bending moment in foot tons.

$f$  = Specified extreme fibre stress in tons per square inch.

$F$  = Factor of safety.

$R_r$  = Equivalent tabular moment of resistance in foot tons.

Then

$$R_r = \frac{7.5 M_r}{f} \text{ or } \frac{M_r F}{4}$$

## MAXIMUM MODULUS OF SECTION.

An alternative and much used method of selecting a section of beam suitable for any system of loading for which the maximum bending moment is known, is offered by the tabulated values of "maximum modulus of section" which for convenience of reference are printed in prominent type.

Any beam or girder will be suitable as regards resistance to flexure provided that it has a "maximum modulus of section" not less than the value ascertained by the appropriate formula of the following, in which:—

$Z$  = tabular section modulus required.

$M$  = maximum bending moment in inch tons.

$M_r$  = " " " " foot tons.

$f$  = extreme fibre stress in tons per square inch.

$W$  = actual load in tons uniformly distributed.

$L$  = effective span in feet.

- (a) For tabular extreme fibre stress of 7.5 tons per square inch, or factor of safety of 4 and any system of loading.

$$Z = 16 M_r \quad \text{or} \quad Z = \frac{M}{7.5}$$

- (b) For any specified extreme fibre stress and any system of loading.

$$Z = \frac{12 M_r}{f} \quad \text{or} \quad Z = \frac{M}{f}$$

- (c) For the tabular conditions of load, support and stress.

$$\begin{aligned} Z &= \frac{WL}{5} \text{ for any span in feet.} \\ &= 2W \text{ for ten feet span.} \\ &= \frac{W}{5} \text{ for one foot span.} \end{aligned}$$

- (d) For the tabular conditions of load and support and any specified extreme fibre stress.

$$Z = \frac{3WL}{2f} \text{ for any span in feet.}$$

# DEFLECTION.

## NOTATION.

- $\delta$  = maximum deflection in inches.  
 $W$  = load in tons.  
 $W_r$  = tabular load in tons.  
 $E$  = modulus of elasticity in tons per square inch.  
 $I$  = moment of inertia in inches<sup>4</sup>.  
 $D$  = overall depth of beam in inches.  
 $F$  = factor of safety.  
 $f$  = extreme fibre stress in tons per square inch.  
 $L$  = span in feet.  
 $l$  = span in inches.  
 $K$  = deflection coefficient.

## DERIVATION OF DEFLECTION COEFFICIENTS.

For the tabular conditions of load and support, the general formulae for the maximum deflection in inches which occurs at the centre of the span is—

$$\delta = \frac{5 W l^3}{384 E I}$$

By substituting equivalent values for  $W$  and  $I$ , expressing the span in feet, assuming  $E = 12,000$  tons per square inch,  $f = 7.5$  tons per square inch for safe loads,  $f = 30$  tons per square inch for breaking loads, the formula becomes—

$$\delta = \frac{3 L^3}{160 D} \text{ for safe load tables,}$$

$$\text{and } \delta = \frac{12 L^3}{160 D} \text{ for breaking load tables.}$$

Each deflection coefficient “ $K$ ” of the tables is equal to:—

$$\frac{3}{160 D} \text{ for safe load tables,}$$

$$\text{or } \frac{12}{160 D} \text{ for breaking load tables.}$$

Therefore the maximum deflection in inches “ $\delta$ ” is equal to:—

$$K L^3 \text{ for safe load tables,}$$

$$\text{or } \frac{K L^3}{F} \text{ for breaking load tables.}$$

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## DEFLECTION—(continued).

### USES OF DEFLECTION COEFFICIENTS.

The tabular deflection coefficients may be used to ascertain the maximum deflection in inches for any specified extreme fibre stress in tons per square inch, or for any specified factor of safety; the nature of the load and support conditions remaining unaltered.

#### SAFE LOAD TABLES.

For specified extreme fibre stress in tons per square inch, $\delta = \frac{KL^2 f}{7.5}$	} or {	For specified factor of safety, $\delta = \frac{4 KL^2}{F}$
--	--------	--

#### BREAKING LOAD TABLES.

For specified extreme fibre stress in tons per square inch. $\delta = \frac{KL^2 f}{30}$	} or {	For specified factor of safety. $\delta = \frac{KL^2}{F}$
---	--------	--

#### Example (1):—

- (a) Required deflection of girder 220 A, 17" × 10", page 22, Part I., under tabular load of 37.4 tons, the span being 20 feet.

#### Answer:—

Deflection coefficient  $K = .001103$ .

$$\therefore \delta = KL^2 = .001103 \times 20^2 = 0.44 \text{ inch} = \frac{1}{2}'' \text{ approximate.}$$

#### Example (2):—

- (b) Required deflection of girder 280 B, 25" × 16", page 30, Part I., for a specified extreme fibre stress of 5 tons per square inch, the span being 30 feet.

#### Answer:—

Deflection coefficient  $K = .00075$ .

$$\therefore \delta = \frac{KL^2 f}{7.5} = \frac{.00075 \times 30^2 \times 5}{7.5} = 0.45 \text{ inch} = \frac{1}{2}'' \text{ approximate.}$$

#### Example (3):—

- (c) Required deflection of angle BSUA25e, 7" × 3½" × ½", page 84, Part I., for a specified factor of safety of 3, the span being 10 feet.

#### Answer:—

Deflection coefficient  $K = .008333$ .

$$\therefore \delta = \frac{KL^2}{F} = \frac{.008333 \times 10^2}{3} = 0.28 \text{ inch} = \frac{1}{4}'' \text{ full.}$$

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## DEFLECTION—(continued).

### SAFE LOAD TABLES.

#### , SYMMETRICAL SECTIONS.

**Ratio of deflection to span for tabular conditions of stress, load, and support.**

If the effective span of a beam is equal to 24 times its depth, the maximum deflection is  $\frac{1}{16}$ th of the span, or approximately  $\frac{1}{16}$ th of an inch per foot of span. The full zigzag lines of the tables indicate this limit for each section.

If a beam supports plaster work such a deflection as the foregoing may be excessive; for this condition it is preferable to limit the effective span to 20 times the depth of the beam, in which case the maximum deflection is  $\frac{1}{16}$ th of the span, or approximately  $\frac{1}{16}$ nd of an inch per foot of span.

A specification may stipulate that the deflection must not exceed a particular ratio of the span such as  $\frac{1}{16}$ th,  $\frac{1}{20}$ th, &c.

Let  $\frac{1}{\gamma}$  = the specified ratio.

Then for the tabular conditions of stress, load and support, the limiting span :—

$$L = \frac{24D \times 320}{\gamma}$$

To obtain the maximum allowable extreme fibre stress in tons per square inch corresponding to a specified ratio of deflection to span, the formula is :—

$$f = \frac{4800D}{\gamma L}$$

To obtain from the tabular load, the maximum allowable load uniformly distributed corresponding to a specified ratio of deflection to span, the formula is :—

$$W = \frac{640D W_T}{\gamma L}$$

or having ascertained  $f$  by the previous formula :—

$$W = \frac{\sqrt{f} W_T}{7.5}$$

or from the moment of inertia :—

$$W = \frac{6400 I}{\gamma L^3}$$

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## DEFLECTION—(continued).

As an example of the foregoing, the 1909 Amendment of the London Building Acts may be cited.

Therein it is stipulated that if the effective span of a beam exceeds 24 times its depth, the deflection must not exceed  $\frac{1}{400}$ th of the span, or  $\frac{1}{\gamma} = \frac{1}{400}$ .

For this particular condition—

$$f = \frac{12 D}{L}$$

and

$$W = \frac{8 D W_f}{5 L} \quad \text{or} \quad W = \frac{16 I}{L^2}$$

### Example:—

Required the maximum allowable extreme fibre stress in tons per square inch, and the maximum load uniformly distributed for girder 106A, 14½" x 10", page 26, Part I., the effective span being 30 feet and the deflection not to exceed  $\frac{1}{400}$ th of the span.

### Answer:—

$$f = \frac{12 \times 14.5}{30} = 5.8 \text{ tons per square inch.}$$

$$W = \frac{8 \times 14.5 \times 29.8}{5 \times 30} = 23 \text{ tons.}$$

$$\text{or } W = \frac{16 \times 1294}{30^2} = 23 \text{ tons.}$$

A beam may have been selected from the tables for a specified load and span, but on calculating the deflection by formula  $\delta = K \times L^2$  it is found that the result exceeds that allowable for some particular condition.

In such a case, a deeper or a heavier beam must be chosen.

If the depth is not restricted the required coefficient is found by dividing the allowable deflection by the square of the span in feet, or:—

$$K = \frac{\delta}{L^2}$$

If the depth cannot be increased a suitable beam may be selected by referring to a tabular load equal to the increased load obtained so:—

$$W_f = \frac{W K L^2}{\delta}$$

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## DEFLECTION—(continued).

### *Example of Unrestricted Depth:—*

Required a beam to support 40 tons uniformly distributed over a span of 20 feet, the deflection not to exceed  $\frac{1}{8}$  inch.

The depth being unrestricted, the required coefficient,

$$K = \frac{\delta}{L^2} = \frac{\cdot 5}{400} = \cdot 00125$$

Therefore any section for which the tabular load for 20 feet span is not less than 40 tons and for which the deflection coefficient is not greater than  $\cdot 00125$  is of sufficient strength.

See pages 16 and 17, Part I.

B.S.B. 29, 20"  $\times$  7 $\frac{1}{2}$ "  $\times$  89 lbs. will support 41.8 tons. The deflection coefficient is  $\cdot 000937$  and the deflection for 40 tons is 0.36 inch.

Similarly see pages 24 and 25, Part I. Girder 143A, Part I., 15 $\frac{1}{2}$ "  $\times$  10"  $\times$  108 lbs. will support 40.7 tons. The deflection coefficient is  $\cdot 00119$  and the deflection for 40 tons is 0.47 inch.

Therefore either of the foregoing sections comply with the conditions.

To show that the deflection coefficient must not exceed  $\frac{\delta}{L^2}$  or  $\cdot 00125$  for the example, unless the tabular load is considerably in excess of the specified load.

See pages 36 and 37, Part I.

Try girder 100B, 13"  $\times$  14"  $\times$  138 lbs.

Tabular load for 20 feet = 40.1 tons.

Deflection coefficient =  $\cdot 001442$ .

Deflection = 0.57 inch.

### *Example of Restricted Depth:—*

Required a beam not more than 16 inches deep to support 17.4 tons uniformly distributed over a span of 26 feet, the deflection not to exceed 0.52 inch.

See pages 16 and 17, Part I.

B.S.B. 27, 18"  $\times$  6"  $\times$  62 lbs. is suitable for depth and load.

Deflection =  $K L^2 = \cdot 001172 \times 26^2$ .

= 0.787 inch.

As this exceeds the specified deflection, an increased tabular load must be referred to.

This increased load =  $\frac{17.4 \times 0.787}{0.52}$   
= 26.4 tons.

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### DEFLECTION—(continued).

See pages 22 and 23, Part I.

Girder 200A, 16" x 10" x 95½ lbs. will support 26·7 tons on 26 feet span.

The deflection coefficient is ·001172, and the deflection for 17·4 tons is 0·516 inch.

Therefore this section complies with all the conditions.

To calculate the maximum deflection in inches for an actual load less than the tabular load the formula is:—

$$\delta = \frac{KL^3W}{W_z}$$

In the last example.

$$\delta = \frac{\cdot001172 \times 26^3 \times 17\cdot4}{26\cdot7}$$
$$= 0\cdot516 \text{ inch.}$$

Particular cases of deflection of unsymmetrical sections, and adaptations of the breaking load tables may be investigated on similar lines to the foregoing.

These are not considered of sufficient importance to treat in detail.

For unsymmetrical sections, substitute for D, twice the distance from the neutral axis to the extreme fibre.

For the breaking load tables substitute  $f = 30$  for  $f = 7\cdot5$ .

It may be pointed out that if a deflection coefficient of the breaking load tables is multiplied by the square of a span in feet, i.e.,  $KL^2$ , the product is a purely imaginary value which must be divided by a factor of safety to produce a deflection within the elastic limit. See notes to Part I.

### DEFLECTION IN PRACTICE.

In practice, and particularly in buildings of the domestic order, which includes offices, clubs and the like, it is frequently observed that the calculated deflection is not realized, but this is due to the fact that the calculated load is not realized in the same proportion.

Another point which may be noted is, that ordinarily a proportion of the loading on a beam is applied gradually as the building operations proceed.

For example, a beam which ultimately will support plaster work, is deflected to some extent by the weight of the floor before the plastering is applied, so that the deflection liable to produce cracks in the plaster is limited to that produced by live load only.

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## DEFLECTION—(continued).

For beams supporting brick walls, the permitted ratios of deflection to span should be as under :—

Length of Span in feet.	Permitted Ratio of Deflection to Span.
Less than 10.	Not exceeding 1/360th.
10 to 15.	1/360th to 1/500th.
15 to 20.	Not exceeding 1/500th.

## WEB BUCKLING.

The maximum allowable load on an unstiffened beam or girder, the reaction at the point or points of support, and the minimum span for uniformly distributed loading are limited by the capacity of the web or webs to resist failure by buckling.

The maximum loads or minimum spans indicated in the tables by dotted zigzag lines have been calculated by the following formulæ which make allowance for the tendency of a web to fail as a thin column by buckling.

- d = net depth of web in inches.
- t = thickness of web in inches.
- n = number of webs (if more than one).
- A = total web area in square inches.
- q = working stress in tons per square inch of web area.
- S = total vertical shear in tons.
- W = maximum allowable load in tons uniformly distributed.
- W<sub>r</sub> = tabular load for 1 foot span.
- L = minimum span in feet corresponding to W,

$$A = d \times t \times n.$$

$$q = 5.5 - .04 \frac{d}{t}$$

$$S = q \times A.$$

S is also the maximum allowable value in tons of a single concentrated load or reaction at any point unless the web is stiffened.

$$W = 2S = 2qA.$$

$$L = \frac{W_r}{2S}$$

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## MINIMUM SPANS AND MAXIMUM LOADS FOR WEB BUCKLING.

### STEEL JOISTS.

Reference Mark.	Section.	Weight per foot in lbs.	Net web area in square inches.	Minimum Span in feet.	Maximum Reaction or Concentrated Load in tons. = S.	Maximum Load Uniformly Distributed in tons. = W.
B.S.B. 30	24 × 7½	100	12·78	10·6	52·1	104·3
B.S.B. 29	20 × 7½	89	10·44	9·2	45·3	90·6
B.S.B. 28	18 × 7	75	8·58	8·5	37·4	74·9
B.S.B. 27	16 × 6	62	7·61	6·6	34·1	68·3
B.S.B. 26	15 × 6	59	6·40	7·3	28·6	57·3
B.S.B. 25	15 × 5	42	5·67	6·0	23·9	47·8
B.S.B. 24	14 × 6a	57	5·95	7·0	27·1	54·1
B.S.B. 23	14 × 6b	46	4·90	7·5	20·9	41·9
B.S.B. 22	12 × 6a	54	4·92	6·7	23·2	46·4
B.S.B. 21	12 × 6b	44	4·08	7·2	18·3	36·6
B.S.B. 20	12 × 5	32	3·68	5·8	15·8	31·6
B.S.B. 19	10 × 8	70	4·56	7·6	22·8	45·5
B.S.B. 18	10 × 6	42	3·24	7·0	15·2	30·4
B.S.B. 17	10 × 5	30	3·07	5·2	14·0	27·9
B.S.B. 16	9 × 7	58	3·68	6·9	18·5	36·9
B.S.B. 15	9 × 4	21	2·35	4·3	10·5	21·0
B.S.B. 14	8 × 6	35	2·86	5·0	14·0	28·1
B.S.B. 13	8 × 5	28	2·27	5·2	10·8	21·6
B.S.B. 12	8 × 4	18	1·92	4·0	8·7	17·4
B.S.B. 11	7 × 4	16	1·48	4·2	6·7	13·5
B.S.B. 10	6 × 5	25	1·88	3·8	9·5	19·0
B.S.B. 9	6 × 4½	20	1·77	3·3	8·8	17·7
B.S.B. 8	6 × 3	12	1·31	2·7	6·2	12·4
B.S.B. 7	5 × 4½	18	1·09	4·2	5·4	10·9
B.S.B. 6	5 × 3	11	·89	3·2	4·2	8·5
B.S.B. 5	4½ × 1½	6½	·72	2·1	3·3	6·6
B.S.B. 4	4 × 3	9½	·68	2·8	3·3	6·7
B.S.B. 3	4 × 1½	5	·58	1·7	2·7	5·4
B.S.B. 2	3 × 3	8½	·42	3·0	2·1	4·3
B.S.B. 1	3 × 1½	4	·38	1·5	1·9	3·8

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## MINIMUM SPANS AND MAXIMUM LOADS FOR WEB BUCKLING.

### STEEL CHANNELS.

Reference Mark.	Section.	Weight per foot in lbs.	Net web area in square inches.	Minimum Span in feet.	Maximum Reaction or Concentrated Load in tons.	Maximum Load Uniformly Distributed in tons.
B.S.C. 27	15 x 4	41.94	7.13	3.9	31.8	63.7
B.S.C. 26	12 x 4	36.47	5.56	3.5	26.1	52.2
B.S.C. 25	12 x 3½	32.88	5.33	3.2	24.8	49.5
B.S.C. 24	12 x 3½	26.10	4.07	3.7	17.7	35.3
B.S.C. 23	11 x 4	33.22	4.82	3.4	22.8	45.6
B.S.C. 22	11 x 3½	29.82	4.61	3.1	21.6	43.2
B.S.C. 21	10 x 4	30.16	4.13	3.3	19.7	39.4
B.S.C. 20	10 x 3½	28.21	4.13	3.0	19.7	39.4
B.S.C. 19	10 x 3½	23.55	3.32	3.4	15.1	30.3
R.S.C. 18	9 x 4	28.55	3.66	3.2	17.7	35.5
B.S.C. 17	9 x 3½	25.39	3.49	2.9	16.8	33.6
B.S.C. 16	9 x 3½	22.27	2.95	3.2	13.7	27.5
R.S.C. 15	9 x 3	19.37	3.00	2.6	14.0	27.9
B.S.C. 14	8 x 4	25.73	3.04	3.1	14.9	29.8
B.S.C. 13	8 x 3½	22.72	2.89	2.8	14.0	28.1
B.S.C. 12	8 x 3	19.30	2.58	2.7	12.3	24.6
B.S.C. 11	8 x 2½	15.12	2.19	2.5	10.1	20.2
B.S.C. 10	7 x 3½	20.23	2.34	2.8	11.5	23.0
B.S.C. 9	7 x 3	17.56	2.23	2.5	10.8	21.7
B.S.C. 8	6 x 3½	17.90	1.84	2.7	9.1	18.3
B.S.C. 7	6 x 3	16.29	1.85	2.4	9.2	18.4
B.S.C. 6	6 x 3	14.49	1.56	2.6	7.6	15.2
B.S.C. 5	6 x 2½	12.04	1.61	2.0	7.8	15.6
B.S.C. 4	5 x 2½	10.38	1.30	1.9	6.4	12.9
R.S.C. 3	4 x 2	7.96	0.79	1.8	4.0	7.9
B.S.C. 2	3½ x 2	6.75	0.70	1.5	3.5	7.1
B.S.C. 1	3 x 1½	5.27	0.58	1.1	3.0	5.9

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### WEB BUCKLING—(continued).

#### Example (1):—

Page 32, Part I., Girder 220 B,  $17'' \times 14'' \times 174$  lbs., composed of 2 steel joists  $16'' \times 6'' \times 62$  lbs. and 2 steel plates  $14'' \times \frac{1}{4}''$ .

(a) Required the maximum allowable value in tons of a single concentrated load.

From Table, page 272, the value of S for 1 -  $16'' \times 6''$  joist is 34.1 tons  
 $\therefore$  S for girder =  $34.1 \times 2 = 68.2$  tons.

#### Example (2):—

(b) Required the minimum span for a uniformly distributed load.

Safe load on 1 foot span:—

$$W_r = 1258 \text{ tons.}$$

$$\therefore L = \frac{W_r}{2S} = \frac{1258}{2 \times 68.2} = 9.2 \text{ feet.}$$

It may also be noted that 9.2 feet is the maximum span for the load of 68.2 tons if concentrated at the centre.

### STIFFENERS.

Stiffeners are necessary where the shearing stress is greater than the allowable value of  $q$  by the formula on page 271.

#### COMPOUND GIRDERS.

Stiffeners are rarely required for compound girders except under heavy concentrated loads and over the supports of girders of very short spans.

#### PLATE GIRDERS.

In plate girder work stiffeners are usually provided under all concentrated loads and over all points of support.

#### GENERAL.

Where uniformly distributed loading produces excessive shear, the stiffeners should be placed at distances apart not exceeding the depth of the girder or at a maximum of 5 feet.

#### PLATE GIRDERS.

Where the shearing stress is less than the allowable stress by formula, stiffeners are commonly provided in plate girders. These may be spaced at convenient distances.

#### SPACING.

In all cases stiffeners should be spaced so as to interfere as little as possible with the uniform pitch of the flange riveting.

For this reason angle sections are more satisfactory than tee sections, although the latter may be used with advantage at a web splice, as they permit of riveting on each side of the joint.

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### STIFFENERS —(continued).

#### DESIGN OF STIFFENERS.

There is no rational method of determining the sections of stiffeners, practice and experience being the only useful guides.

Some authorities suggest that stiffeners may be considered as round-ended struts, free to move in a direction parallel to the web, the allowable stress per square inch being determined by a suitable stanchion formula.

Such a method may answer for stiffeners over reactions or under concentrated loads of considerable amount, but is unlikely to do so for moderate or uniformly distributed loading, owing to the smallness of the resulting sections.

The minimum size of angle used should not be less than  $2'' \times 2'' \times \frac{1}{4}''$ , and in general, the sizes of stiffeners should bear a reasonable proportion to the girders they are employed on.

#### PARALLEL JOISTS AND SEPARATORS.

An economical arrangement for supporting a wall is formed of two or more steel joists parallel and closely adjacent to each other.

In order to insure unity of action, such joists wherever used to support a single load, should be rigidly connected together by separators and bolts.

The separators should be spaced apart throughout the entire length of the joists at distances not exceeding five times the depth over flanges.

They should also be placed immediately over each support and immediately under each concentrated load.

Particulars of cast-iron separators are given on page 317.

#### RIVET PITCH.

The maximum allowable load on a girder and consequently the minimum span are limited by the capacity of the rivets provided in the flanges to resist the horizontal shear.

In view of the heavy plate thicknesses of the compound girders in Part I., this matter has received most careful consideration, and all tabulated safe loads conditional on a rivet pitch closer than 6 inches are printed in italics.

The inclusion of certain sections for which all the safe loads are printed in italics is intentional.

By this means attention is directed automatically to the limits of flange plating as regards the usual rivet pitch, a matter which, as a rule, does not receive sufficient consideration.

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### RIVET PITCH—(continued).

#### TABLES OF MINIMUM SPANS.

The tables of minimum spans in feet for various rivet pitches, pages 50 to 59, Part I., have been calculated for the exact distribution of horizontal shear at the meeting surfaces of the plating and the flange of the component joist, joists or channels as the case may be. The maximum value of the horizontal shear at the point where, owing to the diminution of the bending moment value, the plates might be stopped, has been taken in each case.

The method of using the tables is explained fully in Part I.

#### RIVET SPACING.

The minimum distance from the edge of a rivet or bolt hole to the edge of a member should not be less than the diameter of the hole.

The minimum distance of rivets apart, measured from centre to centre, should not be less than three times the diameter.

The maximum corresponding distance should not exceed sixteen times the thickness of the thinnest member through which they pass.

#### CURTAILMENT OF FLANGE PLATES.

The plates forming the flanges of heavy compound and plate girders may be stopped before reaching the bearings without affecting the carrying capacity.

When it is desired to effect economies in this manner, the consequences of producing a flange not of the same level throughout should be considered.

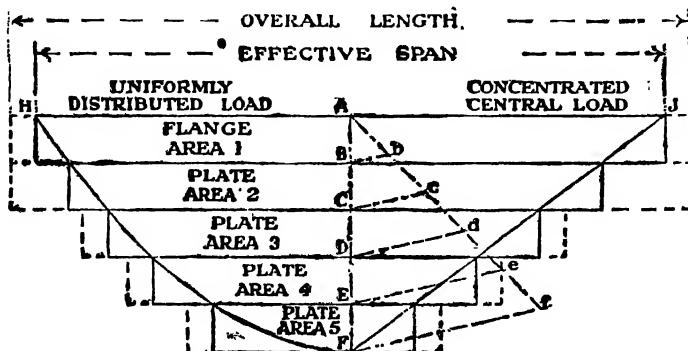
It is obvious for instance, that the top flange plates of a girder for an overhead travelling crane cannot be curtailed with economy, as the rail must be maintained at the same level over its entire length.

The simplest way of ascertaining the points at which the various flange plates may be stopped is the graphic method represented by the following illustration.

The figure shows a bottom flange from the plates, forming which, it is necessary to deduct the loss of area due to rivet holing.

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## CURTAINMENT OF FLANGE PLATES—(continued).



### GRAPHIC METHOD.

The left hand of the figure represents the bending moment diagram for a uniformly distributed load, and the right hand figure that for a concentrated central load.

*In every case the actual bending moment diagram for the system of loading must be drawn. If the loading is symmetrical about the centre of the span it is only necessary to draw one half of the bending moment diagram.*

#### EXPLANATION:—

From A, draw vertical A—F equal to the maximum bending moment, and construct bending moment diagram A F H, or A F J, or otherwise in accordance with system of loading.

From A draw any convenient line A f, and on it mark lengths, A b, b c, c d, d e, e f representing in succession the net areas of the flange, whether joists, channels or angles, and the various plates 2, 3, 4, and 5.

Join f F, and draw parallels e E, d D, c C, and b B intersecting A F at E, D, C and B.

Draw horizontals through B, C, D, E and F.

Theoretically the plates may be stopped at the intersections of the horizontals with the bending moment diagram, as shown by the heavy solid vertical lines.

In practice, to permit of proper riveting, the plates are made about 9 or 12 inches longer at each end as shown by heavy dotted lines.

Area 2, representing the inner plate, is made the full over-all length of the girder.

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### BEARING PLATES.

Bearing plates are necessary wherever the area of the portion of the bottom flange of a beam resting on a wall or pier, is such that the unital pressure transmitted exceeds the safe bearing capacity of the supporting material.

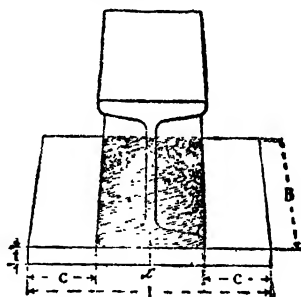
The safe bearing capacity of brick, concrete and other materials are tabulated on page 289.

The superficial area of the bearing plate is determined by dividing the total load transmitted by the safe bearing capacity of the supporting material.

#### DESIGN OF BEARING PLATES.

In order to ensure proper distribution the bearing plate must be of such a thickness that it will not be liable to bend upwards under the load.

The thickness required may be determined by the undernoted formula.



#### NOTATION.

$f$  = working stress in tons per square inch.

$W$  = total load on shaded area in tons.

$l$  = length of plate in inches.

$B$  = breadth of plate in inches.

$t$  = thickness of plate in inches.

$c$  = projection of plate beyond beam in inches.

$$t = \sqrt{\frac{1.5cW}{fB}}$$

$$\text{If } f \text{ is taken at 9 tons per square inch } t = \sqrt{\frac{cW}{6B}}$$

This formula is based on the maximum bending moment of the plate which occurs under the centre of the load at  $x$ .

Bearing plates should not be made of a thickness less than  $\frac{1}{2}$  inch.

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## APPLICATIONS OF THE TABLES OF PART II.

### STANCHIONS. MONCRIEFF FORMULÆ.

Owing to the number of variables on which the calculated strength of a stanchion depends, the tabulated safe concentric loads are not adapted for modifications to the same extent as the tabulated safe loads for beams.

The most convenient general treatment for variations from the tabular end conditions is to ascertain the appropriate working stress per square inch for the required ratio of slenderness and end condition from the table on page 199, Part II.

This value multiplied by the sectional area of the stanchion selected will give the total concentric load.

*Example :—*

Pages 182-183, Part II. Stanchion No. 9 f V; 2 - 3" x 3" x 8" angles battened. Least radius of gyration = 1.09 inches.

Area = 6.72 square inches.

Height = 14 feet.

$$\text{Ratio of slenderness} = \frac{14 \times 12}{1.09} = 155.$$

Required concentric load for "both ends round" and for "both ends fixed."

See page 199, Part II.

(a) Both ends round.

For ratio of slenderness  $\frac{1}{k} = 155$  the working stress, column (1), is 1.56 tons per square inch.

∴ the concentric load is  $6.72 \times 1.56 = 10.5$  tons.

(b) Both ends fixed.

The working stress, column (2), is 4.13 tons per square inch.

∴ the concentric load is  $6.72 \times 4.13 = 27.7$  tons.

### BOTH ENDS ROUND.

For a stanchion having "both ends round," the corresponding safe concentric load can be got directly from the tables by referring to a height in feet equal to twice the actual height.

This rule is accurate only for ratios of slenderness not exceeding  $\frac{106.9}{2} = 53.45$ .

*Example :—*

Pages 132-133, Part II. Stanchion No. 66 K. Steel joist 10" x 6" x 146 lbs. with 2 - 12" x 8" plates on each flange. Height 10 feet.

Required concentric load for "both ends round."

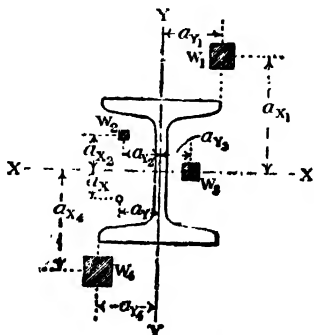
Refer to height of 20 feet and read safe load as 251 tons.

As the ratio of slenderness is  $\frac{10 \times 12}{3} = 40$  this result is accurate.

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## LOCATION OF THE CENTRE OF APPLICATION OF ECCENTRIC LOAD SYSTEMS.

The "arm of eccentricity" or the perpendicular distance from the centre of application of a system of loading to a principal axis of a stanchion is equal to the algebraic sum of the moments of all the loads about the axis, divided by the sum of all the loads.



The moments of the loads on opposite sides of the axis are considered positive and negative respectively, the centre of application of the system being on the same side of the axis as the loads producing the larger sum of moments.

$a_x$  = Arm of eccentricity for axis X—X.

$a_y$  = " " " " Y—Y.

It is required to ascertain the values of  $a_x$  and  $a_y$  for the system of loading shown.

FOR AXIS X—X.

$$a_x = \frac{(W_1 \times a_{x1}) + (W_2 \times a_{x2}) - (W_3 \times a_{x3}) - (W_4 \times a_{x4})}{W_1 + W_2 + W_3 + W_4}.$$

*It may be noted that as  $W_3$  is applied on axis X—X it has no moment about that axis, but its value as a load is taken in the divisor.*

FOR AXIS Y—Y.

$$a_y = \frac{(W_2 \times a_{y2}) + (W_4 \times a_{y4}) - (W_1 \times a_{y1}) - (W_3 \times a_{y3})}{W_2 + W_4 + W_1 + W_3}.$$

FOR A NUMERICAL EXAMPLE LET—

$W_1 = 8$ tons.	$a_{x1} = 6$ inches.	$a_{y1} = 3$ inches.
$W_2 = 5$ "	$a_{x2} = 2$ "	$a_{y2} = 2$ "
$W_3 = 5$ "	$a_{x3} = 0$ "	$a_{y3} = 1$ "
$W_4 = 10$ "	$a_{x4} = 8$ "	$a_{y4} = 3$ "

THEN FOR AXIS X—X.

$$a_x = \frac{8 \times 6 + 5 \times 2 - 10 \times 8}{8 + 5 + 5 + 10} = \frac{58 - 80}{28} = 0.8 \text{ inch nearly.}$$

$a_x$  is on the same side of axis X—X as  $W_4$  since the moment of  $W_4$  is larger than the sum of the moments of  $W_1$  and  $W_2$ .

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CENTRE OF APPLICATION—(continued).

AND FOR AXIS Y—Y.

$$a_y = \frac{5 \times 2 + 10 \times 3 - 8 \times 3 - 5 \times 1}{5 + 10 + 8 + 5} = \frac{40 - 29}{28} \\ = 0.4 \text{ inch nearly.}$$

$a_y$  is on the same side of axis Y—Y as  $W_2$  and  $W_4$ .

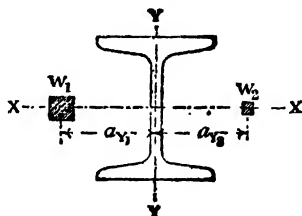
The foregoing method is applicable to any system of loading.

## PARTICULAR CASES.

The following alternative methods are applicable to particular cases.

(a) Two loads of unequal values, concentric relative to axis X—X, and equidistant from axis Y—Y, i.e.  $a_{y1} = a_{y2}$ .

The sum of the loads less their difference, which is equal to twice the value of the lesser load may be treated as concentric, leaving the difference to be treated as eccentric, with an "arm of eccentricity"  $a_y = a_{y1}$  or  $a_{y2}$ .



Then if  $W_e$  = the equivalent concentric load value,

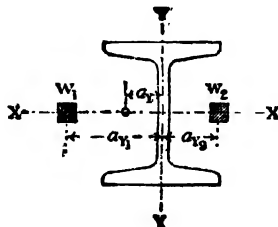
and  $K$  = the eccentricity coefficient,

$$W_e = 2W_s + K(W_1 - W_2).$$

(b) Two loads of equal values, concentric relative to axis X—X but not equidistant from axis Y—Y.

Then, for  $W_1 = W_2$

$$a_y = \frac{a_{y1} - a_{y2}}{2}$$



In both of these cases the treatment for axis X—X is similar.

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## CENTRE OF APPLICATION—(continued).

(c) Two loads of equal or unequal values, each concentric relative to one axis, and eccentric relative to the other axis.

Then for axis  $X-X$ .

Consider  $a_x = a_{x_1}$

and

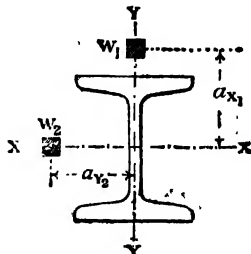
$W_e = W_2 + (W_1 \times K)$ .

And for axis  $Y-Y$ .

Consider  $a_y = a_{y_2}$

and

$W_e = W_1 + (W_2 \times K)$ .



*The methods of using the tabular eccentricity coefficients are explained in the notes to Part II.*

## GENERAL FORMULA FOR ECCENTRIC LOADING.

If a stanchion supports an eccentric load  $W_e$ , having an "arm of eccentricity"  $a$  about a central axis, load  $W_e$  tends to produce bending as shown, tensile stress being developed at the convex side and compressive stress at the concave side.

The maximum compressive stress, which ordinarily is the criterion of strength, occurs at the extreme fibres of the concave side and consists of the direct compressive stress due to the load *plus* the extreme fibre compressive stress due to bending.



### NOTATION.

$W_e$  = actual eccentric load in tons.

$A$  = area of stanchion in square inches.

$I$  = moment of inertia in inches<sup>4</sup> about axis perpendicular to  $a$ .

$k$  = radius of gyration in inches =  $\sqrt{\frac{I}{A}}$ .

$M$  = bending moment in inch tons.

$a$  = arm of eccentricity in inches.

$e$  = perpendicular distance from axis to outer edge of stanchion nearest to load.

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GENERAL FORMULA—(continued).

$f_e$  = extreme fibre compressive stress in tons per square inch due to bending.

$f_c$  = average stress in tons per square inch due to direct compression.

$f$  = maximum extreme fibre compressive stress in tons per square inch  
 $= f_c + f_e$ .

$$M = \begin{cases} W_e \times a \\ \frac{f_e \times I}{e} = \frac{f_e \times Ak^2}{e} \end{cases}$$

$$\therefore f_e \frac{Ak^2}{e} = W_e \times a$$

$$\therefore f_e = \frac{W_e \times a \times e}{Ak^2}$$

$$\text{and } f_c = \frac{W_e}{A}$$

$$\begin{aligned} \therefore f &= f_c + f_e = \frac{W_e}{A} + \frac{W_e \times a \times e}{Ak^2} \\ &= \frac{W_e}{A} \left( 1 + \frac{ae}{k^2} \right) \end{aligned}$$

$W_e$  = equivalent concentric load value in tons. Then

$$W_e = f \times A = W_e \left( 1 + \frac{ae}{k^2} \right)$$

Each tabular eccentricity coefficient is equal to  $1 + \frac{ae}{k^2}$  the value of  $\frac{e}{k^2}$  being the numerical factor of each axial coefficient.

$K$  = eccentricity coefficient.

Then

$$W_e = W_e \times K.$$

### BRACKETS ON STANCHIONS.

From the foregoing it is obvious that for each increase of the value of  $a$ , the "arm of eccentricity," the value of the load which may be supported safely is correspondingly reduced.

For this reason the projection of a bracket should not be more than the minimum necessary to ensure a proper bearing with sufficient space for connecting bolts.

*See types of brackets, Part V.*

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### SLAB CAPS FOR ECCENTRICALLY LOADED SOLID ROUND STEEL STANCHIONS.

The strength of slab caps for eccentrically loaded solid round steel stanchions may be investigated on the following lines:—

The assumption is made that the maximum bending moment will occur at line  $x-x$ , the projection of the cap being considered as a cantilever.

#### NOTATION.

$W$  = eccentric load in tons.

$a$  = distance from centre of application of load to face of stanchion in inches.

$B$  = breadth of cap-plate in inches.

$t$  = thickness of cap-plate in inches.

$M$  = maximum bending moment in inch tons =  $Wa$ .

$f$  = working stress of 7.5 tons per square inch.

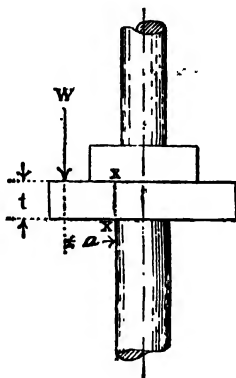
Then

$$M = Wa = \frac{f B t^2}{6}.$$

Hence

$$t = \sqrt{\frac{6 W a}{f B}} = \sqrt{\frac{W a}{1.25 B}}$$

*As a rule the thickness of a slab cap is approximately equal to half the diameter of the stanchion.*



### LATTICED STANCHIONS.

Various formulæ have been evolved for the determination of the stresses in the lattice bars of stanchions, but none of these can be said to be wholly satisfactory for all conditions of height and loading, and lattice bars are generally designed in accordance with the conventions based on sound practical experience.

The following formulæ are from recent publications.

From "*Theory of Structures*" by Prof. Spofford.

The theory of this author is as follows:—

The total stress on a lattice bar is the sum of:—

(a) The compressive stress due to bending occasioned by accidental eccentricity.

(b) The compressive stress due to the shortening of the bar occasioned by the direct compression of the stanchion.

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## LATTICED STANCHIONS—(continued).

The formula for the stress due to bending is based on the assumption that the bending moment in the stanchion may be considered equal to that which would occur if the stanchion were loaded uniformly at right angles to its axis throughout its length.

The value of the assumed load is that which would produce a maximum extreme fibre stress equal to the difference between the allowable compressive stress for  $\frac{1}{k} = 0$ , and the allowable compressive stress for the actual value of  $\frac{1}{k}$ .

This theory is developed to correspond to the bending moment obtained by the straight line formula :—

$$f = 16,000 - \frac{70l}{k}$$

= the allowable compressive stress in lbs. per square inch.

## NOTATION.

$f_m$  = the allowable compressive stress in lbs. per square inch for  $\frac{1}{k} = 0$ .

$f_m = 16,000$  lbs. per square inch.

$f_b$  = the extreme fibre stress in lbs. per square inch.

$$= f_m - f = 16,000 - f.$$

$$= \frac{70l}{k}$$

$M_r$  = the maximum bending moment in inch lbs.

$W$  = the total load in lbs. uniformly distributed.

$A$  = area of stanchion in square inches.

$I$  = relative moment of inertia.

$l$  = height of stanchion in inches.

$e$  = distance from neutral axis to extreme fibres in inches.

$k$  = relative radius of gyration.

$S$  = total shear in lbs.

Hence :—

$$M_r = \frac{f_b \times I}{e} = \frac{W l}{8}$$

$$f_b = \frac{M_r \times e}{I} = \frac{W \times l \times e}{8 I} = \frac{W \times l \times e}{8 \times A \times k^2}$$

$$\therefore \frac{W \times l \times e}{8 \times A \times k^2} = \frac{70l}{k}$$

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## LATTICED STANCHIONS—(continued).

Hence—

$$W = \frac{70 l}{k} \times \frac{8 \times A \times k^3}{l \times e} ;$$

$$= \frac{560 A k}{e}$$

$$\text{But } S = \frac{W}{2}$$

$$\therefore S = \frac{280 A k}{e}$$

or

$$S \text{ in tons} = \frac{A k}{8 e}$$

$P$  = the total compressive stress in one lattice bar in tons.

$\theta$  = the angle of inclination of lattice to the horizontal, not greater than  $45^\circ$ .

$$P = \frac{A k}{16 e} \sec \theta.$$

*This method gives the stress in the end lattice bars, but it is common to use the same size of bars throughout the length of the stanchion.*

The compressive stress due to the shortening of the bar is calculated by the general formulæ of the theory that within the elastic limit unital stress is proportional to unital deformation.

$p$  = the unital compressive stress in one lattice bar occasioned by the direct compression of the stanchion.

$$p = f \sin^2 \theta.$$

$a$  = the area of one lattice bar.

Therefore the total unital compressive stress in one lattice bar due to bending and direct compression.

$$= \frac{A k \sec \theta}{16 e a} + f \sin^2 \theta.$$

$$= \frac{P}{a} + p.$$

*The value so ascertained must not exceed the allowable compressive stress by stanchion formula for the actual value of  $\frac{l}{k}$  for the bar.*

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## LATTICED STANCHIONS—(continued).

*The following is adapted from a widely used American specification:—  
See Proc. Am. Soc. C.E., Vol. 37, Feb. 1911. Discussion on  
Mr. Howard's paper on "Tests of Large Steel Columns."*

### NOTATION.

**S** = transverse shear in tons.

**A** = area of stanchion in square inches.

**k** = relative radius of gyration in inches.

**e** = distance from neutral axis to extreme fibres in inches.

**P** = the total compressive stress in one lattice bar in tons.

**θ** = the angle of inclination of lattice to the horizontal not greater than  $45^\circ$ .

$$S = \frac{A k}{7 e}$$

$$P = \frac{A k}{14 e} \sec \theta.$$

*It will be observed that the above value of P is similar to that of Prof. Spofford's, except for the value of the constant in the denominator.*

*From "Steel Structures," by Clyde T. Morris.*

*The theory of this author is as follows:—*

In a loaded stanchion having both ends fixed, points of contra-flexure are developed.

At these points of contra-flexure the bending moment is zero, and consequently the stress on the cross section is uniform.

Midway between these points the bending moment and the compressive stress at the extreme fibres on the concave side are maximum.

Therefore in a distance equal to  $\frac{1}{4}$ th of the total length of a fixed ended stanchion, the unit stress in the concave side must change from the average to the maximum allowed.

Whence Mr. Morris deduces a formula for the longitudinal increment of stress in one leaf per unit of length of stanchion, and he states that sufficient connection must be provided between the two leaves of the stanchion to transmit this stress.

**A** = the area of the stanchion in square inches.

**$f_m$**  = the allowable compressive stress per square inch by stanchion  
formula when  $\frac{1}{k} = 0$ .

**$f$**  = the allowable compressive stress per square inch by stanchion  
formula for actual value of  $\frac{1}{k}$

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## LATTICED STANCHIONS—(continued).

$L$  = total length of stanchion in feet.

$p$  = change in the total stress in one leaf per unit of length.

$$p = \frac{2A(f_m - f)}{L}$$

From the foregoing an expression for the total compressive stress in one lattice bar may be derived.

$c$  = the horizontal centres of rivets in inches.

$\theta$  = the angle of inclination of lattice to the horizontal, not greater than  $45^\circ$ .

$P$  = the total compressive stress in one lattice bar.

$$P = \frac{A \times c \sec \theta (f_m - f)}{L \times 12}$$

## FOUNDATIONS FOR STANCHIONS.

For the purpose of calculating the total load on stanchion foundations in buildings of more than two storeys in height, excepting buildings of the warehouse class, certain deductions from the live or superimposed loads are permitted, but in every case the dead load must be allowed for in full.

Position of Superimposed Load in Building.	Percentage of each superimposed load to be allowed for on stanchion foundations.
On Roof, - - - - -	100 per cent.
" Top Floor, - - - - -	100 "
" Next Floor below, - - - - -	95 "
" " " " - - - - -	90 "
" " " " - - - - -	85 "
" " " " - - - - -	80 "
and so on, the percentage being decreased by 5 per cent. at each succeeding lower floor until at the 11th floor below the roof the allowable minimum is reached, viz., -	50 "

In all important work the bearing value of the soil should be ascertained by experiment.

On the following page is a table of generally accepted values for the soils, &c., particularized.

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## SAFE PRESSURES ON FOUNDATIONS AND MASONRY. AVERAGE VALUES.

FOUNDATIONS.		Tons per square foot.
Chalk, Hard,	- . . . . .	6
Clay, Ordinary,	- . . . . .	2
" Hard Compact,	- . . . . .	4-5
Earth, Ordinary Firm,	- . . . . .	1
Gravel, Ordinary,	- . . . . .	3
" Compact,	- . . . . .	4-8
Rock, -	- . . . . .	16

MASONRY.		Tons per square foot.
Brick, Ordinary in Cement Mortar,	- . . . . .	5
" Hard (including London Stocks) in Cement Mortar,-	- . . . . .	8
" Blue in Cement Mortar,	- . . . . .	12
Concrete Lime (good),	- . . . . .	8
" Cement (1-2-4),	- . . . . .	12
Freestone, Square Rubble Masonry in Cement,	- . . . . .	8
" Ashlar Masonry in Cement,	- . . . . .	15
" Ashlar Bearing Blocks,	- . . . . .	12
Granite Ashlar Masonry in Cement,	- . . . . .	25
" Ashlar Bearing Blocks,	- . . . . .	20
Granolithic Bearing Blocks,	- . . . . .	12

## STEEL GRILLAGE FOUNDATIONS.

Owing to the low bearing capacity of certain soils it is often essential to distribute a load over a large area.

This is accomplished efficiently and economically by a properly proportioned beam grillage.

The thinness of the foundation so obtainable saves the cost of the deep excavations and large masses of solid concrete otherwise necessary.

A particular advantage of a beam grillage is that by obviating penetration, it may be possible to support a load entirely on a hard crust overlying strata of a soft nature.

A beam grillage consists of one, two or more tiers of steel joists superimposed on a layer of concrete.

Concrete is also rammed into the spaces between the webs of the joists, the flanges being kept sufficiently far apart to allow of this being done properly.

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### STEEL GRILLAGES—(continued).

To avoid displacement during the ramming of the concrete, long bolts with tube separators should pass through the webs of each tier. The flanges of the tiers should also be bolted together.

Joists forming alternate tiers are placed at right angles to each other.

If a grillage consists of two tiers only:—

- (a) the breadth of the upper tier and one side of the stanchion base are equal.
- (b) the length of the upper tier and breadth of the lower tier are equal.

If a grillage consists of more than two tiers:—

- (a) the breadth of an intermediate tier and the length of the next tier above are equal.
- (b) the length of an intermediate tier is a convenient proportion of the length or breadth of the bottom tier.

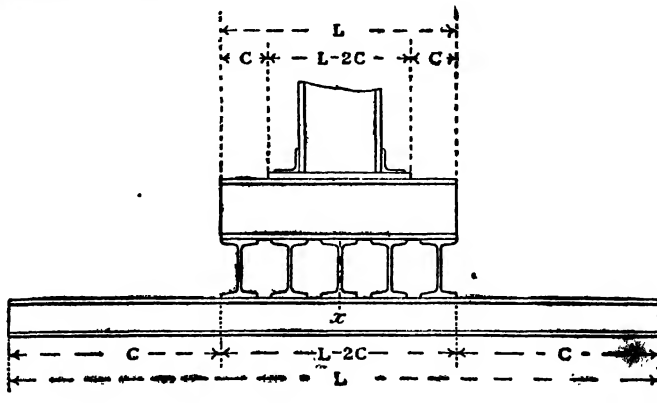
Ordinarily the length and breadth of the bottom tier are approximately equal.

### DESIGN OF BEAM GRILLAGES.

To design a beam grillage it is necessary to know:—

- (a) the total load on the stanchion in tons.
- (b) the size of the stanchion base.
- (c) the safe bearing capacity of the soil in tons per square foot, for which see page 289 *ante*.

The superficial area and consequent overall length and breadth of the bottom tier are determined by dividing the total load by the safe bearing capacity of the soil.



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### STEEL GRILLAGES—(continued).

The diagram represents a three tier grillage.

#### NOTATION.

P = total load in tons on the stanchion.

n = number of joists in the tier.

W = load in tons on one joist of the tier.

$$= P \div n.$$

L = length of the joist in feet.

C = projection in feet.

A suitable section of joist may be selected from the tables in Part I., pages 16 to 19, by either of the two following methods:—

A joist is suitable if,

(a) the tabular load is not less than W for a span equal to 2C feet.

(b) the maximum modulus of section is not less than  $\frac{C \times W}{2.5}$

To prevent web buckling, the load W should not exceed the maximum tabular load for the section of joist selected, or the maximum value of W tabulated on page 272, unless stiffeners are provided.

*Example:—*

Design a beam grillage for a stanchion.

(a) the total load = 200 tons.

(b) the size of stanchion base = 3 feet square.

(c) the safe bearing capacity of the soil = 2 tons per square foot.

Arrange for a three tier grillage.

$$\text{Area of bottom tier} = \frac{200}{2} = 100 \text{ square feet.}$$

$$= 10 \text{ feet long} \times 10 \text{ feet broad.}$$

#### TOP TIER.

Breadth, 3 feet. Length, say 6 feet.

In breadth of 3 feet, 4 beams can be placed with sufficient space for ramming concrete.

$$P = 200. \quad n = 4. \quad \therefore W = \frac{200}{4} = 50 \text{ tons.}$$

$$2C = 6 - 3 = 3 \text{ feet (the equivalent span).}$$

Refer to tables, Part I. Pages 16 and 17.

As the shortest tabulated span is 10 feet, calculate the maximum modulus of section required.

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### STEEL GRILLAGES—(continued).

$$Z = \frac{CW}{2.5} = \frac{1.5 \times 50}{2.5} = 30 \text{ inches}^3.$$

The maximum modulus of section of steel joist  $10'' \times 5'' \times 30$  lbs. is 29.1 inches<sup>3</sup> which is sufficiently near, therefore the section is suitable as regards flexural strength.

For Web buckling refer to page 272. The maximum allowable load is 27.9 tons, therefore 3 sets of cast iron stiffeners instead of the ordinary distance tubes will be required. These may be spaced equally at about 2 feet 9 inches centre to centre.

### MIDDLE TIER.

Breadth 6 feet, length 10 feet.

In breadth of 6 feet, 8 beams can be placed.

$$P = 200, n = 8 \therefore W = \frac{200}{8} = 25 \text{ tons.}$$

$$2C = 10 - 3 = 7 \text{ feet (the equivalent span).}$$

$$Z = \frac{CW}{2.5} = \frac{3.5 \times 25}{2.5} = 35 \text{ inches}^3.$$

The maximum modulus of section of steel joist  $12'' \times 5'' \times 32$  lbs. is 36.6 inches<sup>3</sup>, and the maximum allowable load is 31.6 tons, therefore the section is suitable both as regards flexural strength and web buckling.

### BOTTOM TIER.

Breadth 10 feet, length 10 feet.

In breadth 10 feet, 14 beams can be placed.

$$P = 200, n = 14 \therefore W = \frac{200}{14} = 14.3 \text{ tons.}$$

$$2C = 10 - 6 = 4 \text{ feet (the equivalent span).}$$

Refer to page 18, Part I.

Steel joist  $6'' \times 4\frac{1}{2}'' \times 20$  lbs. will support 14.4 tons on 4 feet span, therefore this section is suitable.

### DERIVATION OF FORMULÆ.

In the foregoing, the assumption is made that the maximum bending moment occurs at the point  $x$  the centre of  $L$ .

Some authorities prefer to treat length  $C$  as a cantilever, with the maximum bending moment occurring at the edge of the tier above.

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### STEEL GRILLAGES—(continued).

Should inequality of bending take place between adjacent tiers or between the top tier and the base plate, the stresses become somewhat indeterminate and under all conditions it is desirable that the worst case likely to occur as represented by maximum bending moment at  $x$ , should be provided for.

*The rules given for the selection of suitable sections are derived from first principles as follows:—*

Consider one beam of the lower tier in diagram.

The load  $W$  is assumed to be distributed uniformly over the length  $L - 2C$ , and the total reaction or upward pressure exerted by the soil is also  $W$  uniformly distributed over the total length  $L$ .

Taking moments about  $x$ .

$$\begin{aligned} M \text{ (foot-tons)} &= \left( \frac{W}{2} \times \frac{L}{4} \right) - \left\{ \frac{W}{2} \times \left( \frac{L - 2C}{4} \right) \right\} \\ &= \frac{W}{8} \{ L - (L - 2C) \} = \frac{2CW}{8} \end{aligned}$$

But  $\frac{2CW}{8}$  is equivalent to the maximum bending moment occurring in a beam supporting a uniformly distributed load over a span of  $2C$  feet, therefore rule No. 1 follows:—

For equilibrium  $M$  must equal  $R$ .

$$M \text{ (inch tons)} = \frac{2CW}{8} \times 12 = 3CW.$$

$$R = fZ \quad \therefore Z = \frac{3CW}{f}$$

and for the tabular conditions for which  $f$  is equal to 7.5 tons per square inch.

$$Z = \frac{3CW}{7.5} = \frac{CW}{2.5}$$

## OVERHEAD TRAVELLING CRANES.

### WHEEL LOADS.

Full particulars of the maximum load on each of the end carriage wheels, and also the centres of the wheels should be obtained from the firm supplying an overhead travelling crane.

If such particulars are not available, the following table will be found useful as a guide, but the data can only be considered as approximately correct as the weights and wheel centres of cranes of the same lifting capacity vary considerably for different makes.

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## ELECTRIC OVERHEAD TRAVELLING CRANES.

### APPROXIMATE PARTICULARS.

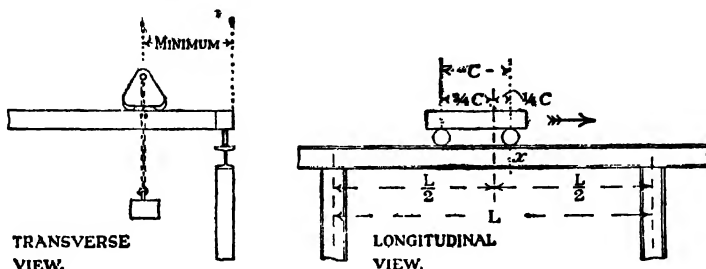
Lift in tons.	DIMENSIONS.				LOADS.				
	Span in feet.	Wheel Base in feet.	Head-room in feet.	End-room in inches.	Total weight of crane in tons <i>excluding lift.</i>	Maximum loads on each end carriage wheel due to proportion of—			
						Weight in tons.	Lift in tons.	Weight and Lift.	
								Actual total in tons.	Equivalent static in tons.
3	25	7·5	4·75	7·5	7·0	1·75	1·5	3·25	4·75
	35	7·5	"	"	8·25	2·07	"	3·57	5·07
	45	9·0	"	"	10·5	2·63	"	4·13	5·63
5	25	7·5	5·5	8·5	8·5	2·13	2·5	4·63	7·13
	35	7·5	"	"	9·75	2·44	"	4·94	7·44
	45	9·0	"	"	11·5	2·88	"	5·35	7·88
10	30	8·0	6·0	9·0	11·25	2·82	5·0	7·82	12·82
	40	8·0	"	"	13·0	3·25	"	8·25	13·25
	50	10·0	"	"	16·25	4·07	"	9·07	14·07
15	30	8·0	6·5	9·5	14·25	3·57	7·5	11·27	18·57
	40	8·0	"	"	16·5	4·13	"	11·63	19·13
	50	10·0	"	"	20·0	5·0	"	12·5	20·0
20	40	9·0	7·0	10·0	21·25	5·32	10·0	15·32	25·32
	50	10·0	"	"	25·0	6·25	"	16·25	26·25
	60	12·0	"	"	28·5	7·13	"	17·13	27·13
25	40	9·0	7·5	10·5	23·5	5·88	12·5	18·38	30·88
	50	10·0	"	"	27·5	6·88	"	19·38	31·88
	60	12·0	"	"	31·5	7·88	"	20·38	32·88

For explanation of equivalent static loads in tons, see page 206.

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## OVERHEAD TRAVELLING CRANES—(continued).

### MAXIMUM LOAD AND BENDING MOMENT.



### MAXIMUM LOAD.

The maximum load on a longitudinal crane girder occurs at each wheel of the end carriage next to which the crab is sustaining the full lift, as shown in transverse view above.

### MAXIMUM BENDING MOMENT.

The maximum bending moment occurs at  $x$  (longitudinal view), when the front wheel has advanced beyond the centre of the span a distance equal to  $\frac{1}{2}C$  (the end carriage wheel centres), unless  $C$  is greater than  $\frac{L}{2}$  in which case the maximum bending moment occurs at the centre of the span when the front wheel is over it.

*For formula for value of maximum bending moment, see page 261.*

### DYNAMIC EFFECT.

The maximum stress produced by a suddenly applied load is **double** that produced by a static load.

### PROPORTION OF STATIC AND DYNAMIC LOADING.

In applying this law to the case of an overhead travelling crane under ordinary working and speed conditions, the weight of the crane itself as it moves along a longitudinal crane girder to the position of maximum bending moment, may be considered as gradually applied or static.

The full lift or capacity of the crane, working at this position for maximum effect is taken as suddenly applied or dynamic.

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### OVERHEAD TRAVELLING CRANES—(continued).

This principle may be used to obtain an equivalent static load to which the ordinary static stress is applicable.

#### EQUIVALENT STATIC LOAD.

##### NOTATION.

$W_o$  = weight of crane.

$W_L$  = lift or capacity of crane.

$W_s$  = equivalent static load on each wheel of an end carriage.

$W_s = \frac{1}{2}W_o + W_L$

See table, page 294.

##### VARIABLE WORKING STRESS.

$f$  = static or ordinary dead load working stress.

$f_s$  = variable working stress regulated by the relative proportions of weight and capacity of crane.

$$f_s = f \left( \frac{W_o + 2 W_L}{W_o + 4 W_L} \right)$$

The foregoing does not take account of impact.

##### IMPACT.

Certain cranes are constructed to lower the burden very rapidly and stop instantaneously, thus producing the effect of impact.

Other analogous cases occur in practice and each of these require to be considered specially.

##### LATERAL FORCES.

Provision should be made for the resistance of the lateral forces exerted on the top flange of a longitudinal crane girder by the cross travel of the crab and by the dragging of loads across the shop floor.

The intensity of these forces must vary according to circumstances, but as a minimum it is usual to provide for a horizontal force of not less than  $\frac{1}{10}$ th of the maximum lifting capacity of the crane.

##### MAXIMUM STANCHION LOAD.

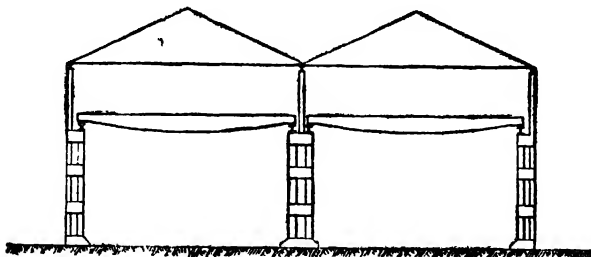
The maximum load on the stanchion occurs when one wheel of the end carriage is immediately over it. The value of the maximum load is equal to the value of the maximum shear for which see page 261.

##### ECCENTRIC LOADING.

The sketch on the following page represents the cross section of a familiar type of engineering shop with an overhead travelling crane in each bay.

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### OVERHEAD TRAVELLING CRANES—(continued).



The stanchions are each composed of two or more members, one of which is continued above the level of the longitudinal crane girders to support the roof.

If the stanchions are considered as compound sections:—

- (a) The system of loading on the side stanchions due to the crane and roof is eccentric about the axis of the stanchion parallel to the longitudinal crane girders, unless the stanchion is symmetrical about that axis and the values of the crane and roof loads are equal. The exception is not likely to occur in practice.
- (b) On the valley stanchions, considering the same axis, greater stresses may be produced by the eccentricity of one crane acting alone, than by the balanced or partially balanced system of loading due to both cranes acting together.

#### CONCENTRIC LOADING.

If the roof loads are very small in comparison with the crane loads, it may be more economical to proportion each member of the stanchion separately of sufficient strength for the particular load it has to support, each load being treated as concentric.

#### DESIGN OF STANCHIONS.

In proportioning stanchions for overhead travelling cranes, the method of obtaining an equivalent static load to which the tabular conditions are applicable will be found convenient.

#### WIND PRESSURE.

The effect of the wind pressure on the roof and side of the building must be allowed for in all cases.

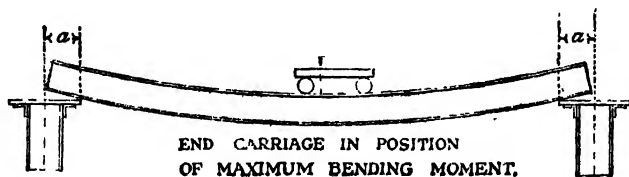
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### OVERHEAD TRAVELLING CRANES—(*continued*).

#### TRANSVERSE AXIS.

It is usual to treat the loading as concentric about the axis of the stanchion parallel to the cross crane girders.

It should be noted, however, that if the longitudinal crane girders are not sufficiently rigid to prevent an appreciable degree of bending, and are not properly connected at the joints, the condition indicated to an exaggerated extent by the sketch may be developed.



Under such circumstances the bearing is transferred to the edge of the cap-plate of the stanchion, and the proportion of the total load equal to the reaction will act with an arm of eccentricity of a maximum value equal to half the length of the cap-plate.

#### MOMENT OF INERTIA.

REFERENCE TO FORMULÆ, Pages 299 to 302.

These tabulated formulæ for the moment of inertia of joists, channels, angles, and tees do not take account of the rounded corners, fillets and tapered flanges of sections rolled to the British standard dimensions.

The resulting values, therefore, do not coincide exactly with the strictly accurate properties in Parts I. and II.

The remaining formulæ are for the various outlines, viz. :—rectangles, triangles, positive and negative sectors of circles into which the profiles of the rolled sections may be divided.

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## PROPERTIES OF VARIOUS SECTIONS.

Section.	Area = A.	Distance to Extreme Fibre = e.	Moments of Inertia.	
			About Central Axis = I.	About Parallel Axis O—O = I <sub>o</sub> .
	2Bs + dt or BD - bd	$\frac{D}{2}$	$I_x$  $\frac{BD^3 - bd^3}{12}$	$I_x + Ae_o^2$ or if $e_o = e_x$ $\frac{BD^3 + bs^3 - b(d+s)^3}{8}$
	2Bs + dt or BD - bd	$\frac{B}{2}$	$I_y$  $\frac{2sB^3 + dt^3}{12}$	$I_y + Ae_o^2$ or if $e_o = e_y$ $\frac{2sB^3 + d\left(\frac{b}{2} + t\right)^3 - d\frac{b^3}{8}}{3}$
	2Bs + dt or BD - bd	$\frac{D}{2}$	$I_x$  $\frac{BD^3 - bd^3}{12}$	$I_x + Ae_o^2$ or if $e_o = e_x$ $\frac{BD^3 + bs^3 - b(d+s)^3}{8}$
	2Bs + dt or BD - bd	$\frac{DB^2 - db^2}{2A}$	$I_y$  $\frac{2se_y^3 + De_y^3 - d(e_y - t)^3}{3}$	$I_y + Ae_o^2$ or if $e_o = e_y$ $\frac{DB^3 - db^3}{3}$ or if $e_o = e_y$ $\frac{2se_y^3 + dt^3}{3}$
	Dt + bs or Bs + dt or BD - bd	$\frac{BD^2 - bd^2}{2A}$	$I_x$  $\frac{te_x^3 + Bex^3 - b'cx - s)}{3}$	$I_x + Ae_o^2$ or if $e_o = e_x$ $\frac{BD^3 - bd^3}{3}$ or if $e_o = e_x$ $\frac{td^3 + bs^3}{3}$

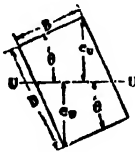
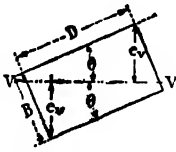
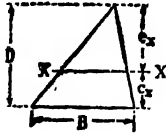
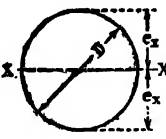
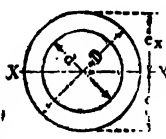
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## PROPERTIES OF VARIOUS SECTIONS.

Section.	Area = A.	Distance to Extreme Fibres = e.	Moments of Inertia.	
			About Central Axis = I.	About Parallel Axis O-O = I <sub>o</sub> .
	$Dt + bs$ or $Bs + dt$ or $BD - bd$	$\frac{BD^3 - bd^3}{2A}$	$I_x$  $\frac{te_x^3 + Bcx^3 - b(cx-s)^3}{8}$	$I_x + Ae_o^2$ or if $e_o = e_x$ $\frac{BD^3 - bd^3}{8}$ or if $e_o = e_x$ $\frac{tD^3 + bs^3}{8}$
	$Dt + bs$ or $Bs + dt$ or $BD - bd$	$\frac{B}{2}$	$I_y$  $\frac{sB^3 + dt^3}{12}$	$I_y + Ae_o^2$ or if $e_o = e_y$ $\frac{sB^3 + d\left(\frac{b}{2} + t\right)^3 - d\frac{b^3}{8}}{8}$
	$B(D-d)$	$\frac{D}{2}$	$I_x$  $\frac{B(D^3 - d^3)}{12}$	$I_x + Ae_o^2$ or if $e_o = e_x$ $\frac{B(D^3 - d^3)}{12} + \frac{Bd^3}{2}$
	$BD$	$\frac{D}{2}$	$I_x$  $\frac{BD^3}{12}$	$I_x + Ae_o^2$ or if $e_o = e_x$ $\frac{BD^3}{8}$
	$DB$	$\frac{B}{2}$	$I_y$  $\frac{DB^3}{12}$	$I_y + Ae_o^2$ or if $e_o = e_y$ $\frac{DB^3}{8}$

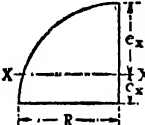
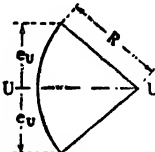
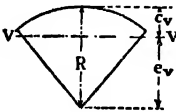
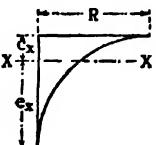
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## PROPERTIES OF VARIOUS SECTIONS.

Section.	Area = A.	Distance to Extreme Fibres = e.	Moments of Inertia.	
			About Central Axis = I.	About Parallel Axis O—O = I <sub>o</sub> .
	BD	$\frac{B \sin \theta + D \cos \theta}{2}$	$I_v = \frac{BD(B^2 \sin^2 \theta + D^2 \cos^2 \theta)}{12}$	$I_v + Ae_o^2$
	DB	$\frac{D \sin \theta + B \cos \theta}{2}$	$I_v = \frac{DB(D^2 \sin^2 \theta + B^2 \cos^2 \theta)}{12}$	$I_v + Ae_o^2$
	$\frac{BD}{3}$	$\frac{2D}{3}$	$I_x = \frac{BD^3}{36}$	$I_x + Ae_o^2$ or if $e_o = e_x$ $\frac{BD^3}{4}$ or if $e_o = e_x$ $\frac{BD^3}{12}$
	$\frac{\pi D^2}{4}$ or $\cdot 7854 D^2$	$\frac{D}{2}$	$I_x = \frac{\pi D^4}{64}$ or $\cdot 0491 D^4$	$I_x + Ae_o^2$ or if $e_o = e_x$ $\frac{5\pi D^4}{64}$ or $\cdot 2455 D^4$
	$\frac{\pi(D^2 - d^2)}{4}$ or $\cdot 7854(D^2 - d^2)$	$\frac{D}{2}$	$I_x = \frac{\pi(D^4 - d^4)}{64}$ or $\cdot 0491(D^4 - d^4)$	$I_x + Ae_o^2$ or if $e_o = e_x$ $\frac{\pi(5D^4 - 4D^2d^2 - d^4)}{64}$

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## PROPERTIES OF VARIOUS SECTIONS.

Section.	Area = A.	Distance to Extreme Fibres = e.	Moments of Inertia.	
			About Central Axis = I.	About Parallel Axis O—O = I <sub>o</sub> .
	$\frac{\pi R^2}{4}$ or *7854R <sup>2</sup>	$R - \frac{4R}{3\pi}$ or *5756R	$I_x$ $\frac{\pi R^4}{16} - \frac{4R^4}{9\pi}$ or *0548R <sup>4</sup>	$I_x + Ae_o^2$ or if $e_o = c_x$ $\frac{\pi R^4}{16}$ or *0626R <sup>4</sup>
	$\frac{\pi R^2}{4}$ or *7854R <sup>2</sup>	$\frac{R}{\sqrt{2}}$ or *7071R	$I_u$ $\frac{\pi R^4}{16} - \frac{R^4}{8}$ or *0713R <sup>4</sup>	$I_u + Ae_o^2$ or if $e_o = e_u$ $\frac{R^4}{8}$
	$\frac{\pi R^2}{4}$ or *7854R <sup>2</sup>	$\frac{4R\sqrt{2}}{3\pi}$ or *6002R	$I_v$ $\frac{\pi R^4}{16} + \frac{R^4}{8} - \frac{8R^4}{9\pi}$ or *0385R <sup>4</sup>	$I_v + Ae_o^2$ or if $e_o = e_v$ $\frac{\pi R^4}{16} + \frac{R^4}{8}$ or *3123R <sup>4</sup>
	$R^2 - \frac{\pi R^2}{4}$ or *2146R <sup>2</sup>	$\frac{R}{6(1 - \frac{\pi}{4})}$ or *7767R	$I_x$ $R^4(\frac{1}{8} - \frac{\pi}{16} - \frac{1}{36 - 9\pi})$ or *6075R <sup>4</sup>	$I_x + Ae_o^2$ or if $e_o = e_x$ $R^4(\frac{1}{8} - \frac{\pi}{16})$ or *1870R <sup>4</sup>

Modulus of Section

$$Z = \frac{I}{e}$$

Radius of Gyration

$$k = \sqrt{\frac{I}{A}}$$

Moment of Resistance

$$R = \frac{fI}{e}$$

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## GENERAL FORMULÆ.

### RELATION OF PROPERTIES AND FORMULÆ FOR FLEXURE.

#### NOTATION.

- I** = moment of inertia about a central axis.  
**I<sub>o</sub>** = " " " " an axis parallel to axis of **I**.  
**c<sub>o</sub>** = perpendicular distance from axis of **I** to axis of **I<sub>o</sub>**.  
**Z** = modulus of section for axis of **I**.  
**e** = perpendicular distance from axis of **I** to extreme fibre of section.  
**k** = radius of gyration for axis of **I**.  
**R** = moment of resistance for axis of **I**.  
**M** = bending moment.  
**f** = intensity of extreme fibre stress.  
**W** = load.  
**l** = effective span.

$$I = Z \times e = A \times k^2 = \frac{M \times e}{f}$$

$$I_o = I + (A \times c_o^2)$$

$$Z = \frac{I}{e} = \frac{M}{f} = \frac{R}{f} \quad k^2 = \frac{I}{A} \quad k = \sqrt{\frac{I}{A}}$$

$$f = \frac{M \times e}{I} = \frac{R \times e}{I} = \frac{M}{Z} = \frac{R}{Z}$$

$$M = R = \frac{f \times I}{e} = f \times Z$$

#### SPECIAL CONDITIONS.

For a beam uniformly loaded and simply supported at each end.

$$W = \frac{8 \times M}{l} = \frac{8 \times f \times I}{l \times e} = \frac{8 \times f \times Z}{l}$$

In addition to above for **I**, **Z**, and **e** in inch units, **W** in tons, **f** value 7.5 tons per square inch, **Mf** in foot tons, and **L** effective span in feet.

$$W = \frac{5 \times I}{L \times e} = \frac{5 \times Z}{L} \quad Z = Mf \times 1.6.$$

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## REFERENCE TO GENERAL FORMULÆ.

These general formulæ are algebraic expressions for the following:--

$$(1) \quad Z = \frac{I}{e} \left\{ \begin{array}{l} \text{Modulus of section} = \text{moment of inertia about a} \\ \text{central axis divided by the perpendicular distance} \\ \text{from the central axis to the extreme fibre of the} \\ \text{section.} \end{array} \right.$$

$$(2) \quad Z = \frac{R}{f} \left\{ \begin{array}{l} \text{Modulus of section} = \text{moment of resistance divided by} \\ \text{extreme fibre stress. } \therefore = \text{moment of resistance for} \\ \text{unit extreme fibre stress of unity, usually 1 ton per} \\ \text{square inch.} \end{array} \right.$$

$$(3) \quad k = \sqrt{\frac{I}{A}} \left\{ \begin{array}{l} \text{Radius of gyration} = \text{square root of moment of inertia} \\ \text{about a central axis divided by the area of the section.} \end{array} \right.$$

$$(4) \quad I_o = I + (A \times Co^2) \left\{ \begin{array}{l} \text{Moment of inertia about an axis O - O parallel to the} \\ \text{central axis of } I = \text{the moment of inertia about the} \\ \text{central axis plus the product of the area of the section} \\ \text{by the square of the perpendicular distance between} \\ \text{the axes.} \end{array} \right.$$

$$(5) \quad R = M \left\{ \begin{array}{l} \text{For equilibrium the moment of resistance of a section} \\ \text{must equal the maximum bending moment due to the} \\ \text{external forces.} \end{array} \right.$$

## PROPERTIES OF COMPOUND SECTIONS.

The moments of inertia and other properties of any desired compound section may be ascertained by using the undernoted tabulated values for the simple sections.

Moments of inertia, Part I., pages 17 to 99.

Net moments of inertia, Part IV., page 325.

Moments of inertia of plates, Part IV., pages 325-327.

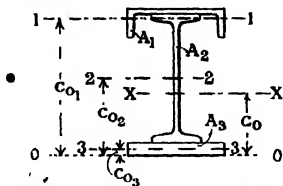
Positions of central axis, Part V., pages 390-402.

## MOMENT OF INERTIA OF AN UNSYMMETRICAL COMPOUND SECTION.

It is first necessary to ascertain the position of the central axis of the compound section about which the moment of inertia is required.

# REDPATH, BROWN & CO., LIMITED.

## MOMENT OF INERTIA—(continued).



As an example, consider the unsymmetrical compound section above.

### NOTATION.

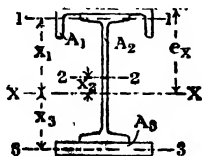
- $X-X$  = the central axis of the compound section, the position of which is required.  
 $O-O$  = a convenient axis parallel to axis  $X-X$ . Usually the most convenient position for this axis coincides with an outer edge of the compound section.  
 $A_1, A_2, A_3$  = the component areas.  
 $1-1, 2-2, 3-3$  = the central axes of  $A_1, A_2, A_3$ , parallel to axes  $X-X$  and  $O-O$ .  
 $c_{01}, c_{02}, c_{03}$  = the perpendicular distances from  $1-1, 2-2, 3-3$  to  $O-O$ .  
 $c_0$  = the required perpendicular distance from axis  $O-O$  to axis  $X-X$ .

Then

$$c_0 = \frac{A_1 \times c_{01} + A_2 \times c_{02} + A_3 \times c_{03}}{A_1 + A_2 + A_3}$$

Or: the distance of the centre of area of a plane figure from any point in its plane is equal to the sum of the moments of all the component areas about the point divided by the sum of the component areas.

Having ascertained the position of axis  $X-X$  the moment of inertia of the compound section about that axis may now be calculated.



- $I_1, I_2, I_3$  = the moments of inertia of the component areas about the central axes parallel to  $X-X$ .  
 $x_1, x_2, x_3$  = the perpendicular distances from  $1-1, 2-2, 3-3$ , to  $X-X$ .

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## MOMENT OF INERTIA—(continued).

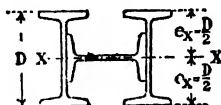
$I_x$  = the required moment of inertia of the compound section about axis X—X.

Then

$$I_x = I_1 + I_2 + I_3 + (A_1 \times x_1^2) + (A_2 \times x_2^2) + (A_3 \times x_3^2)$$

Or: the moment of inertia of a compound section about a central axis is equal to the sum of the moments of inertia of the component areas about their respective parallel central axes plus the sum of the products of each component area into the square of the distance from its central axis to the central axis of the compound section. The foregoing is an application of formula (4) page 304.

## MOMENT OF INERTIA AND MODULUS OF SECTION OF A COMPOUND SECTION ABOUT AN AXIS OF SYMMETRY.



If a central axis such as X—X of a compound section coincides with a central axis of each of the component areas or sections it is an axis of symmetry; there are no  $x$  distances to consider and:—

$$I_x = I_1 + I_2 + I_3$$

$$Z = \frac{2I_x}{D}$$

## DEDUCTIONS FOR RIVET HOLES.

The calculations for ascertaining the exact deductions to be made for rivet holes are extremely laborious.

In practice sufficient accuracy is attained by deducting one rivet hole from each flange for reeled riveting, or two rivet holes from each flange for straight riveting.

The tables of the net moments of inertia of joists and channels and fractional plate widths given on pages 325-327 of this part will be found most convenient for this purpose.

**REDPATH, BROWN & CO., LIMITED.**

**MISCELLANEOUS  
STRUCTURAL  
TABLES.**

# REDPATH, BROWN & CO., LIMITED.

## SHEARING AND BEARING VALUES FOR BOLTS AND RIVETS.

Shearing Values at { 5 tons per square inch for Single Shear. Bearing Values at 10  
8.75 " " " Double Shear. tons per square inch.

Diameter of Bolt or Rivet. Inches.	Area of Bolt or Rivet. Square Inches.	Shearing Values.		Bearing Values. Tons.							
		Single Shear. Tons.	Double Shear. Tons.	Thickness of Plate in Inches.							
				1/4	1/8	3/8	7/16	1/2	5/8	3/4	7/8
1/4	0.491	245	429	625	781						
3/8	1.104	552	966	937	1172	1406					
1/2	1.963	981	1717	1250	1562	1875	2187				
5/8	3.068	1534	2684	1562	1953	2344	2734	3125			
3/4	4.418	2209	3866	1875	2344	2812	3281	3750	4687	5625	
7/8	6.013	3006	5261	2187	2734	3281	3828	4375	5469	6562	
1	7.854	3927	6872	2500	3125	3750	4375	5000	6250	7500	8750

Shearing Values at { 5.5 tons per square inch for Single Shear. Bearing Values at 11  
9.625 " " " Double Shear. tons per square inch.

Diameter of Bolt or Rivet. Inches.	Area of Bolt or Rivet. Square Inches.	Shearing Values.		Bearing Values. Tons.							
		Single Shear. Tons.	Double Shear. Tons.	Thickness of Plate in Inches.							
				1/4	1/8	3/8	7/16	1/2	5/8	3/4	7/8
1/4	0.491	270	472	687	859						
3/8	1.104	607	1062	1031	1289	1547					
1/2	1.963	1079	1889	1375	1719	2062	2406				
5/8	3.068	1687	2953	1719	2148	2578	3008	3437			
3/4	4.418	2430	4252	2062	2578	3094	3609	4125	5156	6187	
7/8	6.013	3307	5787	2406	3008	3609	4211	4812	6015	7219	
1	7.854	4320	7559	2760	3437	4125	4812	5500	6875	8250	9625

# REDPATH, BROWN & CO., LIMITED.

## SHEARING AND BEARING VALUES FOR BOLTS AND RIVETS.

Shearing Values at { 6 tons per square inch for Single Shear. Bearing Values at 12  
10.5 " " " Double Shear. tons per square inch.

Diameter of Bolt or Rivet.  Inches.	Area of Bolt or Rivet, Square Inches.	Shearing Values.		Bearing Values. Tons.							
		Single Shear. Tons.	Double Shear. Tons.	Thickness of Plate in Inches.							
				$\frac{1}{8}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$
$\frac{1}{8}$	0491	294	515	750	937						
$\frac{3}{16}$	1101	662	1159	1125	1406	1687					
$\frac{1}{4}$	1963	1178	2061	1500	1875	2250	2625				
$\frac{5}{16}$	3068	1841	3221	1875	2344	2812	3281	3750			
$\frac{3}{8}$	4418	2651	4639	2250	2812	3375	3937	4500	5625	6750	
$\frac{7}{16}$	6013	3608	6313	2625	3281	3937	4594	5250	6562	7875	
1	7854	4712	8247	3000	3750	4500	5250	6000	7500	9000	1050

In the above tables double shear is taken at 1.75 times single shear, and the bearing value at twice single shear.

Bearing Values printed in ordinary type are greater than double shear for the corresponding diameters. In these cases the shearing values are the determining factors.

Bearing Values printed in prominent type are greater than single and less than double shear for the corresponding diameters, so that in case of

(a) Single shear, the shearing value is the criterion.

(b) Double shear, the bearing value " "

Bearing Values printed in italics are less than single shear. In these cases the bearing values are the determining factors.

# REDPATH, BROWN & CO., LIMITED.

## WHITWORTH STANDARD BOLTS AND NUTS.



Hexagon Head and Nut and Round Neck.

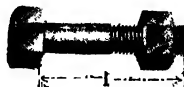
APPROXIMATE WEIGHT IN LBS. OF ONE BOLT AND NUT.

Length inches.	DIAMETER IN INCHES.										
	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{3}{8}$	$1\frac{1}{2}$
1	·081	·092	·200	·389	·613						
$1\frac{1}{4}$	·083	·096	·207	·380	·628						
$1\frac{1}{2}$	·085	·100	·214	·391	·644	·989					
$1\frac{3}{4}$	·086	·103	·221	·403	·661	1·011					
$1\frac{7}{8}$											
$2\frac{1}{8}$	·038	·108	·228	·414	·677	1·033	1·439				
$2\frac{1}{4}$	·040	·112	·236	·425	·694	1·055	1·518				
$2\frac{1}{2}$	·042	·116	·243	·436	·709	1·077	1·546	2·118			
$2\frac{3}{4}$	·044	·120	·250	·448	·725	1·099	1·575	2·177			
2	·046	·124	·257	·458	·742	1·121	1·604	2·214	2·051	3·851	
$2\frac{1}{4}$	·048	·128	·264	·470	·758	1·143	1·632	2·251	2·096	3·904	
$2\frac{1}{2}$	·050	·132	·271	·481	·773	1·165	1·661	2·286	3·041	3·959	5·032
$2\frac{3}{4}$	·051	·136	·279	·492	·790	1·187	1·690	2·323	3·085	4·014	5·097
$2\frac{7}{8}$											
$3\frac{1}{8}$	·053	·140	·286	·504	·806	1·209	1·718	2·359	3·131	4·067	5·161
$3\frac{1}{4}$	·056	·149	·300	·526	·838	1·252	1·776	2·432	3·220	4·176	5·290
3	·060	·156	·315	·549	·871	1·296	1·833	2·463	3·309	4·285	5·420
$3\frac{1}{4}$	·064	·164	·329	·571	·903	1·340	1·891	2·577	3·398	4·393	5·549
$3\frac{1}{2}$											
$3\frac{3}{4}$	·067	·173	·343	·592	·935	1·384	1·928	2·650	3·489	4·502	5·675
$3\frac{7}{8}$	·071	·180	·358	·616	·968	1·429	2·005	2·722	3·578	4·610	5·807
	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{3}{8}$	$1\frac{1}{2}$
For each additional inch length of Shank add.	·014	·031	·055	·085	·123	·176	·218	·276	·381	·418	·491
TO ASCERTAIN THE WEIGHT OF ANY BOLT AND NUT HAVING OTHER FORMS OF HEAD AND NUT, TAKE THE WEIGHT AS SHOWN ABOVE AND ADD AS FOLLOWS:—											
For square head,	·0009	·0033	·0079	·0156	·0274	·0427	·0637	·0843	·1225	·1655	·2154
For square nut,	·0011	·0038	·0091	·0180	·0306	·0493	·0735	·0974	·1410	·1912	·2487

# REDPATH, BROWN & CO., LIMITED.

## WHITWORTH STANDARD BOLTS AND NUTS.

Hexagon Head and Nut and Round Neck.



APPROXIMATE WEIGHT IN LBS. OF ONE BOLT AND NUT.

Length Inches.	DIAMETER IN INCHES.										
	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{3}{8}$	$1\frac{1}{2}$
4	.074	.189	.372	.638	1.000	1.473	2.062	2.795	3.667	4.718	5.936
4½	.078	.197	.386	.680	1.032	1.517	2.119	2.867	3.756	4.827	6.065
4¾	.081	.206	.401	.683	1.064	1.561	2.177	2.941	3.846	4.936	6.195
5	.085	.213	.415	.705	1.097	1.605	2.234	3.013	3.936	5.044	6.223
5½	.089	.221	.429	.727	1.129	1.649	2.292	3.085	4.025	5.152	6.452
5¾	.092	.229	.444	.750	1.161	1.692	2.348	3.156	4.114	5.261	6.581
6	.096	.237	.458	.772	1.193	1.736	2.406	3.230	4.204	5.369	6.710
6½	.099	.245	.472	.795	1.226	1.786	2.464	3.304	4.293	5.478	6.839
7	.103	.254	.487	.817	1.258	1.824	2.501	3.376	4.383	5.587	6.968
7½	.110	.269	.515	.863	1.323	1.913	2.635	3.521	4.562	5.804	7.227
8	.117	.285	.544	.907	1.388	2.001	2.749	3.666	4.741	6.020	7.485
8½	.124	.302	.573	.952	1.462	2.087	2.865	3.812	4.920	6.238	7.743
9		.319	.601	.997	1.517	2.176	2.973	3.957	5.098	6.455	8.001
9½		.334	.631	1.041	1.581	2.264	3.094	4.102	5.278	6.671	8.259
10			.659	1.086	1.646	2.353	3.208	4.248	5.456	6.888	8.517
10½			.687	1.131	1.710	2.441	3.323	4.393	5.686	7.106	8.775
11				1.177	1.774	2.528	3.437	4.531	5.814	7.322	9.083
11½				1.221	1.839	2.616	3.553	4.684	5.993	7.529	9.292
12					1.904	2.704	3.667	4.828	6.172	7.757	9.550
						2.798	3.782	4.973	6.351	7.973	9.808
							3.896	5.119	6.531	8.191	10.066
	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{3}{8}$	$1\frac{1}{2}$
For each additional inch length of Shank add.	.014	.031	.055	.085	.123	.176	.218	.276	.381	.418	.491
TO ASCERTAIN THE WEIGHT OF ANY BOLT AND NUT HAVING OTHER FORMS OF HEAD AND NUT, TAKE THE WEIGHT AS SHOWN ABOVE AND ADD AS FOLLOWS:—											
For Square Head,	.0009	.0033	.0079	.0156	.0274	.0427	.0637	.0843	.1225	.1655	.2154
For Square Nut,	.0011	.0038	.0091	.0180	.0306	.0493	.0725	.0974	.1410	.1912	.2487

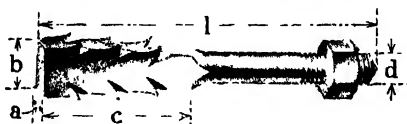
# REDPATH, BROWN & CO., LIMITED.

## **BOLTS AND NUTS.**

Whitworth's Standard Sizes.

Diameter of Bolt. Inches.		Number of Threads per inch.	Diameter at Bottom of Thread.	Distances in inches over		Thickness of Bolt Head. Inches.	Sectional Area at Bottom of Thread. Sq. inches.
Fraction.	Decimal.			Flats.	Corners.		
$\frac{1}{16}$	.250	20	.186	.525	.606	.219	.027
$\frac{1}{8}$	.375	16	.295	.709	.819	.328	.068
$\frac{3}{16}$	.500	12	.393	.919	1.061	.437	.121
$\frac{1}{4}$	.625	11	.508	1.101	1.271	.547	.203
$\frac{5}{16}$	.750	10	.622	1.301	1.502	.656	.304
$\frac{3}{8}$	.875	9	.733	1.479	1.707	.766	.422
1	1.000	8	.840	1.670	1.928	.875	.554
$1\frac{1}{16}$	1.125	7	.942	1.860	2.148	.984	.697
$1\frac{1}{8}$	1.250	7	1.067	2.048	2.365	1.094	.894
$1\frac{3}{8}$	1.375	6	1.161	2.215	2.557	1.203	1.059
$1\frac{1}{2}$	1.500	6	1.286	2.413	2.787	1.312	1.300

## **LEWIS BOLTS AND NUTS.**



Diameter d inches.	Overall Length l inches.	Taper Length c inches.	Base.		Diameter d inches.	Overall Length l inches.	Taper Length c inches.	Base.	
			a inches.	b inches.				a inches.	b inches.
$\frac{1}{8}$	5	3	$\frac{1}{8}$	$\frac{7}{8}$	1	8	4½	1	1½
$\frac{1}{4}$	6	3	$\frac{1}{4}$	1½	1½	9	5	1½	1½
$\frac{3}{8}$	6	3	$\frac{3}{8}$	1½	1½	10	6	1½	2½
$\frac{1}{2}$	7	3½	$\frac{1}{2}$	1½	1½	12	7	1½	2½

## **GAS TUBING.**

Approximate Weights and Sizes.

Nominal Bore, inches,	-	-	-	-	2	1	1½	1½	2
Thickness { S.W.G.,	-	-	-	-	11	10	9	8	8
Inches, -	-	-	-	-	.116	.128	.144	.160	.160
No. of Threads per inch (Whitworth),	-	-	-	-	14	11	11	11	11
Weight per foot in lbs.,	-	-	-	-	1.18	1.79	2.52	2.97	4.48

# REDPATH, BROWN & CO., LIMITED.

## HEXAGON COUPLING BOXES.

Screwed Right and Left Hand Thread.

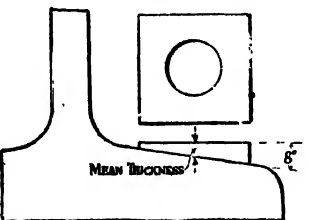


Diameter. Inches.	Length of Box. Inches.	Length of Ends. Inches.	Diameter. Inches.	Length of Box. Inches.	Length of Ends. Inches.
$\frac{1}{8}$	$1\frac{1}{2}$	$2\frac{1}{2}$	1	7	8
$\frac{1}{4}$	$2\frac{1}{2}$	3	$1\frac{1}{4}$	8	8
$\frac{3}{8}$	3	$4\frac{1}{2}$	$1\frac{1}{2}$	9	9
$\frac{1}{2}$	4	6	$1\frac{3}{4}$	$9\frac{1}{2}$	10
$\frac{5}{8}$	$4\frac{1}{2}$	6	$1\frac{7}{8}$	10	12
$\frac{3}{4}$	5	6	$1\frac{7}{8}$	11	12
$\frac{7}{8}$	6	7	2	12	12
$1\frac{1}{8}$	$6\frac{1}{2}$	7			

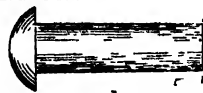
## ORDINARY WASHERS.

Diameter of Bolt. Inches.	Outside Diameter of Washer. Inches.	Thickness of Washer. Inches.	Weight per 100. Lbs.	Diameter of Bolt. Inches.	Outside Diameter of Washer. Inches.	Thickness of Washer. Inches.	Weight per 100. Lbs.
$\frac{1}{8}$	$1\frac{1}{4}$	$\frac{1}{8}$	$2\frac{1}{2}$	1	$2\frac{1}{4}$	$\frac{3}{16}$	14
$\frac{1}{4}$	$1\frac{3}{8}$	$\frac{1}{8}$	4	$1\frac{1}{8}$	$2\frac{3}{8}$	$\frac{3}{16}$	$17\frac{1}{2}$
$\frac{3}{8}$	$1\frac{7}{8}$	$\frac{1}{8}$	$5\frac{1}{2}$	$1\frac{1}{4}$	$2\frac{7}{8}$	$\frac{3}{16}$	$21\frac{1}{2}$
$\frac{1}{2}$	$1\frac{7}{8}$	$\frac{1}{8}$	$7\frac{1}{2}$	$1\frac{3}{8}$	$2\frac{7}{8}$	$\frac{3}{16}$	26
				$1\frac{1}{2}$	$3\frac{1}{8}$	$\frac{3}{16}$	$30\frac{1}{2}$

## SQUARE BEVELLED WASHERS.

	Diameter of Bolt. Inches.	Side of Square. Inches.	Mean Thickness. Inches.	Weight per 100. Lbs.
	$\frac{1}{8}$ $\frac{1}{4}$ $\frac{3}{8}$ $\frac{1}{2}$ 1	$1\frac{1}{8}$ $1\frac{3}{8}$ $1\frac{5}{8}$ $1\frac{7}{8}$ $2\frac{1}{8}$	$\frac{3}{16}$ $\frac{3}{16}$ $\frac{3}{16}$ $\frac{3}{16}$ $\frac{3}{16}$	6 $8\frac{1}{2}$ $10\frac{1}{2}$ $15\frac{1}{2}$ 20

# REDPATH, BROWN & CO., LIMITED.



## STEEL CUP HEADED RIVETS.

Approximate Weights in Lbs. per 100.

Length Inches.	DIAMETERS IN INCHES.						Length Inches.
	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1	
1	4'39	9'74					1
1 $\frac{1}{8}$	5'28	10'43					1 $\frac{1}{8}$
1 $\frac{1}{4}$	5'67	11'13					1 $\frac{1}{4}$
1 $\frac{3}{8}$	6'06	11'82					1 $\frac{3}{8}$
1 $\frac{1}{2}$	6'45	12'52	21'20	32'86			1 $\frac{1}{2}$
1 $\frac{5}{8}$	6'84	13'22	22'28	34'42			1 $\frac{5}{8}$
1 $\frac{3}{4}$	7'23	13'91	23'37	35'98			1 $\frac{3}{4}$
1 $\frac{7}{8}$	7'62	14'61	24'46	37'55			1 $\frac{7}{8}$
2	8'01	15'30	25'55	39'11	56'43	77'88	2
2 $\frac{1}{8}$	8'40	16'00	26'63	40'68	58'56	80'66	2 $\frac{1}{8}$
2 $\frac{1}{4}$	8'79	16'70	27'72	42'44	60'68	83'44	2 $\frac{1}{4}$
2 $\frac{3}{8}$	9'18	17'39	28'81	43'81	62'81	86'22	2 $\frac{3}{8}$
2 $\frac{1}{2}$	9'57	18'09	29'90	45'37	64'94	89'01	2 $\frac{1}{2}$
2 $\frac{5}{8}$	9'96	18'79	30'98	46'54	67'07	91'79	2 $\frac{5}{8}$
2 $\frac{3}{4}$	10'35	19'48	32'07	48'50	69'20	94'57	2 $\frac{3}{4}$
2 $\frac{7}{8}$	10'74	20'18	33'15	50'07	71'33	97'35	2 $\frac{7}{8}$
3		20'87	34'24	51'63	73'46	100'13	3
3 $\frac{1}{8}$		21'57	35'32	53'19	75'59	102'91	3 $\frac{1}{8}$
3 $\frac{1}{4}$		22'27	36'41	54'76	77'72	105'69	3 $\frac{1}{4}$
3 $\frac{3}{8}$		22'96	37'50	56'32	79'85	108'47	3 $\frac{3}{8}$
3 $\frac{1}{2}$			38'59	57'89	81'98	111'26	3 $\frac{1}{2}$
3 $\frac{5}{8}$			39'67	59'45	84'11	114'04	3 $\frac{5}{8}$
3 $\frac{3}{4}$			40'76	61'02	86'23	116'82	3 $\frac{3}{4}$
3 $\frac{7}{8}$			41'85	62'58	88'36	119'60	3 $\frac{7}{8}$
4				64'15	90'49	122'38	4
4 $\frac{1}{8}$				65'71	92'62	125'16	4 $\frac{1}{8}$
4 $\frac{1}{4}$				67'28	94'75	127'94	4 $\frac{1}{4}$
4 $\frac{3}{8}$				68'84	96'88	130'72	4 $\frac{3}{8}$
4 $\frac{1}{2}$					99'01	133'51	4 $\frac{1}{2}$
4 $\frac{5}{8}$					101'14	136'29	4 $\frac{5}{8}$
4 $\frac{3}{4}$					103'27	139'07	4 $\frac{3}{4}$
4 $\frac{7}{8}$					105'40	141'85	4 $\frac{7}{8}$
5					107'53	144'63	5
5 $\frac{1}{8}$					109'66	147'41	5 $\frac{1}{8}$
5 $\frac{1}{4}$					111'78	150'19	5 $\frac{1}{4}$
5 $\frac{3}{8}$						152'97	5 $\frac{3}{8}$
5 $\frac{1}{2}$						155'76	5 $\frac{1}{2}$
5 $\frac{5}{8}$						158'54	5 $\frac{5}{8}$
5 $\frac{3}{4}$						161'32	5 $\frac{3}{4}$
5 $\frac{7}{8}$						164'10	5 $\frac{7}{8}$
6						166'88	6
Weight Per 100 Heads, lbs.	1'76	4'17	8'15	14'08	22'86	33'38	Weight Per 100 Heads, lbs.
For each additional Inch Length of Shank add per 100.	3'13	5'67	8'70	12'52	17'03	22'25	For each additional Inch Length of Shank add per 100.

REDPATH, BROWN & CO., LIMITED.

## MACHINE RIVETING.

### Lengths of Rivets for Varying Grips.



Grip. Inches.	Diameters in Inches.				Grip. Inches.	Diameters in Inches.			
	1	2	3	4		1	2	3	4
	Lengths in Inches.					Lengths in Inches.			
1	1 1/8	2 1/8	3 1/8	4 1/8	1	1 1/8	2 1/8	3 1/8	4 1/8
1 1/4	1 1/4	2 1/4	3 1/4	4 1/4	1 1/4	1 1/4	2 1/4	3 1/4	4 1/4
1 1/2	1 1/2	2 1/2	3 1/2	4 1/2	1 1/2	1 1/2	2 1/2	3 1/2	4 1/2
1 3/4	1 3/4	2 3/4	3 3/4	4 3/4	1 3/4	1 3/4	2 3/4	3 3/4	4 3/4
2	2	3	4	5	2	2	3	4	5
2 1/4	2 1/4	3 1/4	4 1/4	5 1/4	2 1/4	2 1/4	3 1/4	4 1/4	5 1/4
2 1/2	2 1/2	3 1/2	4 1/2	5 1/2	2 1/2	2 1/2	3 1/2	4 1/2	5 1/2
2 3/4	2 3/4	3 3/4	4 3/4	5 3/4	2 3/4	2 3/4	3 3/4	4 3/4	5 3/4
3	3	4	5	6	3	3	4	5	6
3 1/4	3 1/4	4 1/4	5 1/4	6 1/4	3 1/4	3 1/4	4 1/4	5 1/4	6 1/4
3 1/2	3 1/2	4 1/2	5 1/2	6 1/2	3 1/2	3 1/2	4 1/2	5 1/2	6 1/2
3 3/4	3 3/4	4 3/4	5 3/4	6 3/4	3 3/4	3 3/4	4 3/4	5 3/4	6 3/4
4	4	5	6	7	4	4	5	6	7
4 1/4	4 1/4	5 1/4	6 1/4	7 1/4	4 1/4	4 1/4	5 1/4	6 1/4	7 1/4
4 1/2	4 1/2	5 1/2	6 1/2	7 1/2	4 1/2	4 1/2	5 1/2	6 1/2	7 1/2
4 3/4	4 3/4	5 3/4	6 3/4	7 3/4	4 3/4	4 3/4	5 3/4	6 3/4	7 3/4
5	5	6	7	8	5	5	6	7	8

**For Hand Riveting deduct  $\frac{1}{4}$ -in. from above Lengths.**

# REDPATH, BROWN & CO., LIMITED.

## GALVANISED CORRUGATED SHEETS.

SIZES AND APPROXIMATE WEIGHTS IN LBS. PER SHEET.

Size, Length x Breadth, Feet.	GAUGE.					Size, Length x Breadth, Feet.	GAUGE.				
	18	20	22	24	26		18	20	22	24	26
4 x 2	20	16	13½	11	8½	8 x 2	41½	32	26½	22	16½
4½ x "	23½	18	15	12½	9½	8½ x "	44½	34	28½	23½	17½
5 x "	26	20	17	14	10½	9 x "	47	36	30	25	18½
5½ x "	27½	22	18½	15½	11½	10 x "	52	40	34	28	21
6 x "	31½	24	20½	16½	12½						
6½ x "	34	26	21½	17½	13½	6 x 2½	37½	29	24½	20	15
7 x "	36½	28	23½	19	14½	7 x "	44	34	28½	23	17½
7½ x "	39½	30	25	20½	16½	8 x "	50	38½	32½	26½	20
1 sq. foot.	2·41	1·85	1·55	1·29	0·95	100 sq. feet.*	241	185	155	129	95

ADDITIONAL WEIGHTS IN LBS. PER SQUARE.\*

For single side lap and 6-in. end lap, . . .  
For double " " 6-in. " " . . .

GAUGE.				
18	20	22	24	26
32·1	24·7	20	17·2	12·7
62·3	47·8	40	33·3	24·5

## GALVANISED SHEET FITTINGS.

SIZES AND APPROXIMATE WEIGHTS IN LBS. PER GROSS AND SQUARE.\*



### HOOK BOLTS.

Length, 1 inches, . . . .	3½	4	4½	5
Diameter, inches, . . . .	¾	¾	¾	¾
Weight per gross, . . . .	18·7	24·9	20·4	28·0
Weight per square (4 per sq. yd.), . . . .	5·8	7·7	6·3	8·6
			6·9	9·9
				7·7
				11·5

### WASHERS.

For ½-in. diameter bolts

WEIGHTS per Gross.

1·96 lbs. 5·22 lbs.

WEIGHTS per Square.

·81 lb. - 3·0 lbs.



### ROOFING SCREWS.



½-in diameter.

2½	3	Length, inches.	2½	3
5·1	5·9	Weight per Gross.	5·3	7·0
2·1	2·4	Weight per Square.	2·2	2·9

### SHEETING BOLTS.

### CUP HEAD RIVETS.

½-in. diameter.

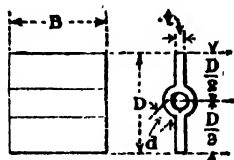
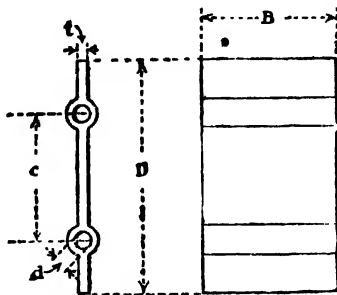
¾	1½	1½	Length, inches.	¾	¾	¾
3·6	4·7	5·1	Weight per Gross.	2·0	2·15	2·3
1·5	2·0	2·2	Weight per Square.	·81	·87	·93



\*1 "square" = 100 square feet.

# REDPATH, BROWN & CO., LIMITED.

## CAST IRON SEPARATORS.



Steel Joist.		Bolts.			Cast Iron Separator.					Steel Joist. Size, inches.
Size, inches.	Centres of Webs, inches.	Size, inches.	Weight, lbs.	Type.	Size, D x B x t inches.	Holes.		Weight, lbs.	Weight, per inch width, lbs.	
						Centres, inches c.	Dia., inches d.			
24 x 7½	8.10	9½ x 7	2.48	1	19½ x 7½ x 5	12	7	26½	3.53	24 x 7½
20 x 7½	8.10	9½ x 7	2.48	1	16½ x 7½ x 5	10	6	22	2.94	20 x 7½
18 x 7	7.55	9½ x 7	2.39	1	13½ x 7 x 5	9	6	17½	2.50	18 x 7
16 x 6	6.55	8 x 6	1.52	1	12 x 6 x 5	6½	6	12	2.00	16 x 6
15 x 6	6.50	7½ x 6	1.50	1	10½ x 6 x 5	6	6	11	1.83	15 x 6
15 x 5	5.42	6½ x 5	1.35	1	10½ x 5 x 5	6	6	9	1.80	15 x 5
14 x 6a	6.50	7½ x 6	1.50	1	10½ x 6 x 5	5½	6	11	1.83	14 x 6a
14 x 6b	6.40	7½ x 6	1.48	1	10½ x 6 x 5	5½	6	11	1.83	14 x 6b
12 x 6a	6.50	7½ x 6	1.50	1	8½ x 6 x 5	5	6	9	1.50	12 x 6a
12 x 6b	6.40	7½ x 6	1.48	1	8½ x 6 x 5	5	6	9	1.50	12 x 6b
12 x 5	5.60	6½ x 5	1.37	1	8½ x 5½ x 5	5	5	8	1.52	12 x 5
10 x 6	6.40	7½ x 6	1.48	1	7 x 6 x 5	4	6	8	1.33	10 x 6
10 x 5	5.61	6½ x 5	1.37	1	7 x 5½ x 5	4	6	7	1.33	10 x 5
9 x 4	4.55	5½ x 4	1.23	1	7 x 4½ x 5	4	4	5½	1.29	9 x 4
8 x 6	6.44	7½ x 6	1.48	1	5 x 6 x 5	3	6	6½	1.08	8 x 6
8 x 5	5.60	6½ x 5	1.37	1	5 x 5½ x 5	3	6	5½	1.05	8 x 5
8 x 4	4.53	5½ x 4	1.23	1	5 x 4½ x 5	3	4	4½	1.06	8 x 4
7 x 4	4.50	5½ x 4	1.21	2	5 x 4½ x 5	3	4	3½	.82	7 x 4
6 x 5	5.41	6½ x 5	1.35	2	3 x 5 x 5	3	3	3	.60	6 x 5
6 x 4½	5.12	6½ x 5	1.31	2	3 x 4½ x 5	3	3	3	.66	6 x 4½
6 x 3	3.51	4½ x 4	1.08	2	3 x 3½ x 5	3	3	2	.61	6 x 3

# REDPATH, BROWN & CO., LIMITED.

## SECTIONAL AREAS OF STEEL ANGLES IN SQUARE INCHES.

Breadths of Flanges added in inches.	THICKNESS OF ANGLE IN INCHES.										
	$\frac{3}{16}$	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	1
$\frac{1}{8}$	.43	.56									
$\frac{1}{4}$	.48	.63									
$\frac{3}{8}$	.53	.69									
$\frac{1}{2}$	.57	.75	.92								
$\frac{5}{8}$	.62	.81	1.00	1.17							
$\frac{3}{4}$	.67	.88	1.07	1.27							
4	.72	.94	1.15	1.36							
$4\frac{1}{2}$	.76	1.00	1.23	1.45							
$4\frac{3}{4}$	.81	1.06	1.31	1.55	1.78						
5	.86	1.13	1.39	1.64	1.88	2.13					
$5\frac{1}{2}$	.90	1.19	1.46	1.73	2.00	2.25	2.50				
$5\frac{3}{4}$	.95	1.25	1.54	1.83	2.11	2.38	2.64				
6	1.00	1.31	1.62	1.92	2.21	2.50	2.78	3.05			
$6\frac{1}{2}$	1.04	1.38	1.70	2.02	2.32	2.63	2.92	3.20			
7	1.09	1.44	1.78	2.11	2.43	2.75	3.06	3.36	3.65	3.94	
$7\frac{1}{2}$	1.14	1.50	1.86	2.20	2.54	2.88	3.20	3.52	3.82	4.13	
$7\frac{3}{4}$	1.18	1.56	1.93	2.30	2.65	3.00	3.34	3.67	4.00	4.31	
8	1.23	1.63	2.01	2.39	2.76	3.13	3.48	3.83	4.17	4.50	
$8\frac{1}{2}$	1.28	1.69	2.09	2.48	2.87	3.25	3.62	3.98	4.34	4.69	
$8\frac{3}{4}$	1.32	1.75	2.17	2.58	2.98	3.38	3.76	4.14	4.51	4.88	
9	1.37	1.81	2.25	2.67	3.09	3.50	3.90	4.30	4.68	5.06	
$9\frac{1}{2}$	1.42	1.88	2.32	2.77	3.20	3.63	4.04	4.45	4.86	5.26	
$9\frac{3}{4}$	1.47	1.94	2.40	2.86	3.31	3.75	4.18	4.61	5.03	5.44	
10	1.51	2.00	2.48	2.95	3.42	3.88	4.32	4.77	5.20	5.63	
$10\frac{1}{2}$	1.56	2.06	2.56	3.05	3.53	4.00	4.46	4.92	5.37	5.81	
$10\frac{3}{4}$	1.61	2.13	2.64	3.14	3.64	4.13	4.61	5.08	5.54	6.00	
$11$			2.72	3.23	3.75	4.25	4.75	5.23	5.71	6.19	7.11
$11\frac{1}{2}$			2.79	3.33	3.86	4.38	4.89	5.39	5.89	6.38	7.33
$11\frac{3}{4}$			2.87	3.42	3.97	4.50	5.03	5.55	6.06	6.57	7.55
12			2.95	3.52	4.07	4.68	5.17	5.70	6.23	6.75	7.77
$12\frac{1}{2}$			3.61	4.18	4.75	5.31	5.86	6.40	6.94	7.48	8.90
$12\frac{3}{4}$			3.70	4.29	4.88	5.45	6.02	6.57	7.13	7.68	9.25
$13$			3.80	4.40	5.00	5.59	6.17	6.75	7.31	7.87	9.50
$13\frac{1}{2}$			3.89	4.51	5.13	5.73	6.33	6.92	7.50	8.04	9.76
$13\frac{3}{4}$			3.98	4.62	5.25	5.87	6.48	7.09	7.69	8.26	10.00
14			4.08	4.73	5.38	6.01	6.64	7.26	7.88	8.48	10.25
$14\frac{1}{2}$			4.17	4.84	5.50	6.15	6.80	7.43	8.06	8.68	10.50
$14\frac{3}{4}$			4.27	4.95	5.68	6.29	6.95	7.61	8.25	8.92	10.75
15			4.36	5.06	5.75	6.43	7.11	7.78	8.44	9.13	11.00

### WEIGHTS OF STEEL ANGLES IN LBS. PER LINEAL FOOT.

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# REDPATH, BROWN & CO., LIMITED.

## FLAT ROLLED STEEL.

Sectional Areas in Square Inches.

Breadth in Inches.	THICKNESS IN FRACTIONS OF AN INCH.																
	$\frac{1}{8}$	$\frac{3}{16}$	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$1\frac{1}{16}$	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{3}{8}$	$1\frac{1}{2}$	2
1	250	312	375	437	500	562	625	687	750	812	875	937	1000	1062	1125	1187	1250
1 1/8	281	351	421	492	562	632	703	773	843	913	984	1054	1125	1195	1265	1335	1406
1 1/4	312	390	468	546	625	703	781	859	937	1015	1094	1172	1250	1328	1406	1484	1562
1 1/2	345	429	515	601	687	773	859	945	1031	1117	1203	1289	1375	1461	1547	1633	1719
1 3/4	375	468	562	656	750	843	937	1031	1125	1219	1312	1406	1500	1594	1687	1781	1875
2	406	507	609	710	812	913	1015	1117	1219	1321	1423	1525	1627	1729	1831	1933	2035
2 1/8	437	546	656	765	875	984	1094	1203	1312	1423	1531	1640	1749	1857	1966	2075	2184
2 1/4	469	586	703	820	937	1054	1172	1289	1406	1525	1643	1761	1879	1997	2115	2233	2351
2 1/2	500	625	750	875	1000	1125	1250	1375	1500	1625	1750	1875	2000	2125	2250	2375	2500
2 3/4	531	664	796	929	1061	1193	1325	1457	1589	1721	1853	1985	2117	2249	2381	2513	2645
3	562	703	843	984	1125	1265	1406	1547	1687	1828	1968	2109	2249	2389	2529	2669	2809
3 1/8	594	743	890	104	118	133	147	161	175	189	203	217	231	245	259	273	287
3 1/4	625	781	937	109	125	141	156	171	186	201	216	231	246	261	276	291	306
3 1/2	656	820	984	115	131	147	164	180	196	212	228	244	260	276	292	308	324
3 3/4	687	859	103	120	137	154	172	189	206	223	240	257	274	291	308	325	342
4	719	898	107	126	143	160	178	195	213	230	248	265	283	300	317	335	352
4 1/8	750	937	112	131	150	167	186	203	221	239	257	275	293	311	329	347	365
4 1/4	781	101	122	142	162	182	203	223	243	263	283	303	323	343	363	383	403
4 1/2	812	101	122	142	162	182	203	223	243	263	283	303	323	343	363	383	403
4 3/4	843	101	122	142	162	182	203	223	243	263	283	303	323	343	363	383	403
5	875	101	122	142	162	182	203	223	243	263	283	303	323	343	363	383	403
5 1/8	906	101	122	142	162	182	203	223	243	263	283	303	323	343	363	383	403
5 1/4	937	101	122	142	162	182	203	223	243	263	283	303	323	343	363	383	403
5 1/2	100	125	150	175	200	225	250	275	300	325	350	375	400	425	450	475	500
5 3/4	106	132	159	185	212	238	265	291	318	344	371	397	423	449	475	501	527
6	112	140	168	197	225	253	281	309	337	365	393	421	449	477	505	533	561
6 1/8	119	148	178	207	237	266	296	325	355	384	413	442	471	500	529	558	587
6 1/4	125	156	187	218	250	281	312	343	375	406	437	468	499	530	561	592	623
6 1/2	131	164	196	229	262	293	325	357	389	421	453	485	517	549	581	613	645
6 3/4	137	171	206	240	275	309	343	377	411	445	479	513	547	581	615	649	683
7	143	179	215	251	287	323	359	395	431	467	503	539	575	611	647	683	719
7 1/8	150	187	225	262	300	337	375	413	450	487	525	562	600	637	675	712	750
7 1/4	156	197	235	273	312	350	388	426	464	502	540	578	616	654	692	730	768
7 1/2	162	203	244	284	325	366	407	447	487	527	567	607	647	687	727	767	807
7 3/4	175	218	262	306	350	394	437	481	525	568	612	656	700	743	787	831	875
8	187	234	281	328	375	423	469	516	562	608	654	700	746	792	838	884	930
8 1/8	200	250	300	350	400	450	500	550	600	650	700	750	800	850	900	950	1000
8 1/4	212	265	318	371	425	478	531	584	637	690	743	796	849	902	955	1008	1061
8 1/2	225	281	337	393	450	506	562	618	674	730	786	842	898	954	1010	1066	1122
8 3/4	237	296	355	415	475	535	595	654	713	772	831	890	949	1008	1067	1126	1185
9	250	312	375	437	500	562	625	687	750	812	875	937	1000	1062	1125	1187	1250
9 1/8	262	328	393	459	525	591	656	721	787	852	917	982	1047	1112	1177	1242	1307
9 1/4	275	343	412	481	550	618	687	755	823	891	959	1027	1095	1163	1231	1299	1367
9 1/2	287	360	431	503	575	646	717	788	859	929	1000	1070	1140	1210	1280	1350	1420
10	300	375	450	525	600	675	750	825	900	975	1050	1125	1200	1275	1350	1425	1500
10 1/8	312	390	468	546	625	703	781	859	937	1015	1094	1172	1250	1328	1406	1484	1562
10 1/4	325	406	488	569	650	731	812	893	974	1054	1135	1215	1295	1375	1455	1535	1615
10 1/2	337	423	507	591	675	759	843	927	1011	1094	1177	1260	1343	1426	1509	1592	1675
10 3/4	350	437	523	609	694	779	864	949	1033	1117	1201	1285	1369	1453	1537	1621	1705
11	362	450	537	625	712	799	886	973	1059	1145	1231	1317	1403	1489	1575	1661	1747
11 1/8	375	468	556	643	731	818	905	992	1079	1166	1253	1340	1427	1514	1601	1688	1775
11 1/4	387	481	570	657	745	832	919	1006	1093	1180	1267	1354	1441	1528	1615	1702	1789
12	400	500	600	700	800	900	1000	1100	1200	1300	1400	1500	1600	1700	1800	1900	2000

# REDPATH, BROWN & CO., LIMITED.

## FLAT ROLLED STEEL.

Weight per Lineal Foot in Lbs.

Width in Inches.	THICKNESS IN FRACTIONS OF AN INCH.															Width in Inches.
	$\frac{1}{16}$	$\frac{1}{8}$	$\frac{3}{16}$	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$1$	
1	.85	1.06	1.28	1.49	1.70	1.91	2.13	2.34	2.55	2.76	2.98	3.19	3.40			1
1 $\frac{1}{4}$	.90	1.20	1.43	1.67	1.91	2.15	2.39	2.63	2.87	3.11	3.35	3.59	3.83			1 $\frac{1}{4}$
1 $\frac{1}{2}$	1.06	1.33	1.59	1.86	2.13	2.39	2.66	2.92	3.19	3.45	3.72	3.98	4.25			1 $\frac{1}{2}$
1 $\frac{3}{4}$	1.17	1.46	1.75	2.05	2.34	2.63	2.92	3.21	3.51	3.80	4.09	4.38	4.68			1 $\frac{3}{4}$
2	1.28	1.59	1.91	2.23	2.55	2.87	3.19	3.51	3.83	4.14	4.46	4.78	5.10			2
2 $\frac{1}{4}$	1.38	1.73	2.07	2.42	2.76	3.11	3.45	3.80	4.14	4.49	4.83	5.18	5.53			2 $\frac{1}{4}$
2 $\frac{1}{2}$	1.49	1.86	2.23	2.60	2.98	3.35	3.72	4.09	4.46	4.83	5.21	5.58	5.95			2 $\frac{1}{2}$
2 $\frac{3}{4}$	1.59	1.99	2.39	2.79	3.19	3.59	3.98	4.39	4.78	5.18	5.58	5.98	6.38			2 $\frac{3}{4}$
3	1.70	2.13	2.55	2.98	3.40	3.83	4.25	4.68	5.10	5.53	5.95	6.38	6.80			3
3 $\frac{1}{4}$	1.81	2.26	2.71	3.16	3.61	4.07	4.52	4.97	5.42	5.88	6.32	6.78	7.23			3 $\frac{1}{4}$
3 $\frac{1}{2}$	1.91	2.39	2.87	3.35	3.83	4.30	4.78	5.26	5.74	6.22	6.69	7.17	7.65			3 $\frac{1}{2}$
3 $\frac{3}{4}$	2.02	2.52	3.03	3.53	4.04	4.54	5.05	5.55	6.06	6.57	7.07	7.57	8.08			3 $\frac{3}{4}$
4	2.13	2.66	3.19	3.72	4.25	4.78	5.31	5.84	6.38	6.91	7.44	7.97	8.50			4
4 $\frac{1}{4}$	2.23	2.79	3.35	3.91	4.46	5.02	5.58	6.14	6.69	7.26	7.81	8.37	8.93			4 $\frac{1}{4}$
4 $\frac{1}{2}$	2.34	2.92	3.51	4.09	4.68	5.26	5.84	6.43	7.01	7.60	8.18	8.77	9.35			4 $\frac{1}{2}$
4 $\frac{3}{4}$	2.44	3.06	3.67	4.28	4.89	5.50	6.11	6.72	7.33	7.95	8.55	9.17	9.78			4 $\frac{3}{4}$
5	2.55	3.19	3.83	4.46	5.10	5.74	6.38	7.01	7.65	8.29	8.93	9.56	10.20			5
5 $\frac{1}{4}$	2.76	3.45	4.14	4.83	5.53	6.22	6.91	7.60	8.29	8.98	9.67	10.36	11.05			5 $\frac{1}{4}$
5 $\frac{1}{2}$	2.98	3.72	4.46	5.21	5.95	6.70	7.44	8.18	8.93	9.67	10.41	11.16	11.90			5 $\frac{1}{2}$
5 $\frac{3}{4}$	3.19	3.98	4.78	5.58	6.38	7.17	7.97	8.77	9.56	10.36	11.16	11.95	12.75			5 $\frac{3}{4}$
6	3.40	4.25	5.10	5.95	6.80	7.65	8.50	9.35	10.20	11.05	11.90	12.75	13.60			6
6 $\frac{1}{4}$	3.61	4.52	5.42	6.32	7.23	8.13	9.03	9.93	10.84	11.74	12.64	13.56	14.45			6 $\frac{1}{4}$
6 $\frac{1}{2}$	3.83	4.78	5.74	6.69	7.65	8.61	9.56	10.52	11.48	12.43	13.39	14.34	15.30			6 $\frac{1}{2}$
6 $\frac{3}{4}$	4.04	5.05	6.06	7.07	8.08	9.08	10.09	11.10	12.11	13.12	14.13	15.14	16.15			6 $\frac{3}{4}$
7	4.25	5.31	6.38	7.44	8.50	9.56	10.63	11.69	12.75	13.81	14.88	15.94	17.00			7
7 $\frac{1}{4}$	4.46	5.58	6.69	7.81	8.93	10.04	11.16	12.27	13.39	14.50	15.62	16.73	17.85			7 $\frac{1}{4}$
7 $\frac{1}{2}$	4.68	5.84	7.01	8.18	9.35	10.52	11.69	12.86	14.03	15.19	16.36	17.53	18.70			7 $\frac{1}{2}$
7 $\frac{3}{4}$	4.89	6.11	7.33	8.55	9.78	11.00	12.22	13.44	14.66	15.88	17.11	18.33	19.55			7 $\frac{3}{4}$
8	5.10	6.38	7.65	8.93	10.20	11.48	12.75	14.03	15.30	16.58	17.85	19.13	20.40			8
8 $\frac{1}{4}$	5.53	6.91	8.29	9.67	11.05	12.43	13.81	15.19	16.58	17.96	19.34	20.72	22.10			8 $\frac{1}{4}$
8 $\frac{1}{2}$	5.96	7.44	8.93	10.41	11.89	13.39	14.88	16.36	17.85	19.34	20.83	22.31	23.80			8 $\frac{1}{2}$
8 $\frac{3}{4}$	6.38	7.97	9.56	11.16	12.75	14.34	15.94	17.53	19.13	20.72	22.31	23.91	25.50			8 $\frac{3}{4}$
9	6.80	8.50	10.20	11.90	13.60	15.30	17.00	18.70	20.40	22.10	23.80	25.50	27.20			9
9 $\frac{1}{4}$	7.23	9.03	10.84	12.63	14.45	16.26	18.06	19.87	21.68	23.48	25.29	27.10	28.90			9 $\frac{1}{4}$
9 $\frac{1}{2}$	7.65	9.50	11.48	13.50	15.50	17.51	19.51	21.51	23.51	25.51	27.51	29.51	31.51			9 $\frac{1}{2}$
9 $\frac{3}{4}$	8.08	10.00	12.11	14.13	16.15	18.17	20.19	22.21	24.23	26.24	28.26	30.28	32.30			9 $\frac{3}{4}$
10	8.50	10.63	12.75	14.88	17.00	19.13	21.25	23.38	25.50	27.63	29.75	31.88	34.00			10
10 $\frac{1}{4}$	8.93	11.16	13.39	15.62	17.85	20.08	22.31	24.54	26.78	29.01	31.24	33.47	35.70			10 $\frac{1}{4}$
10 $\frac{1}{2}$	9.35	11.60	14.03	16.36	18.70	21.04	23.38	25.71	28.05	30.39	32.73	35.06	37.40			10 $\frac{1}{2}$
10 $\frac{3}{4}$	9.78	12.22	14.66	17.11	19.55	22.00	24.44	26.88	29.33	31.77	34.21	36.66	39.10			10 $\frac{3}{4}$
11	10.20	12.75	15.30	17.85	20.40	22.95	25.50	28.05	30.60	33.15	35.70	38.25	40.80			11
11 $\frac{1}{4}$																11 $\frac{1}{4}$
11 $\frac{1}{2}$																11 $\frac{1}{2}$
12																12

Values for additional width of  $\frac{1}{4}$ "

$\frac{1}{4}$	213	266	319	372	425	478	531	584	638	691	744	797	850	$\frac{1}{4}$
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# REDPATH, BROWN & CO., LIMITED.

## FLAT ROLLED STEEL

Weight per Lineal Foot in Lbs. (continued).

Width in Inches.	THICKNESS IN FRACTIONS OF AN INCH.																Width in Inches.
	$\frac{1}{8}$	$\frac{3}{16}$	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{7}{8}$	$\frac{15}{16}$	1			
12 $\frac{1}{2}$	10.62	13.28	15.94	18.50	21.25	23.90	26.56	29.22	31.88	34.54	37.20	39.84	42.50	12 $\frac{1}{2}$	12 $\frac{1}{2}$		
13	11.05	13.81	16.58	19.34	22.10	24.86	27.63	30.39	33.15	35.91	38.68	41.44	44.20	13	13		
13 $\frac{1}{2}$	11.47	14.34	17.21	20.08	22.95	25.82	28.69	31.56	34.42	37.29	40.16	43.03	45.90	13 $\frac{1}{2}$	13 $\frac{1}{2}$		
14	11.90	14.88	17.86	20.83	23.80	26.78	29.75	32.73	35.70	38.68	41.65	44.63	47.60	14	14		
14 $\frac{1}{2}$	12.32	15.41	18.49	21.57	24.65	27.73	30.81	33.90	36.97	40.05	43.13	46.22	49.30	14 $\frac{1}{2}$	14 $\frac{1}{2}$		
15	12.75	15.94	19.13	22.31	25.50	28.69	31.88	35.06	38.25	41.44	44.63	47.81	51.00	15	15		
15 $\frac{1}{2}$	13.17	16.47	19.76	23.05	26.35	29.64	32.94	36.23	39.52	42.82	46.11	49.40	52.70	15 $\frac{1}{2}$	15 $\frac{1}{2}$		
16	13.60	17.00	20.40	23.80	27.20	30.60	34.00	37.40	40.80	44.20	47.60	51.00	54.40	16	16		
16 $\frac{1}{2}$	14.02	17.53	21.04	24.54	28.05	31.55	35.06	38.57	42.07	45.58	49.09	52.59	56.10	16 $\frac{1}{2}$	16 $\frac{1}{2}$		
17	14.45	18.06	21.68	25.29	28.90	32.51	36.13	39.74	43.35	46.96	50.58	54.19	57.80	17	17		
17 $\frac{1}{2}$	14.87	18.59	22.31	26.03	29.75	33.47	37.19	40.91	44.62	48.34	52.07	55.78	59.50	17 $\frac{1}{2}$	17 $\frac{1}{2}$		
18	15.30	19.13	22.95	26.78	30.60	34.43	38.25	42.08	45.90	49.73	53.55	57.38	61.20	18	18		
18 $\frac{1}{2}$	15.72	19.66	23.59	27.52	31.45	35.39	39.31	43.24	47.17	51.11	55.04	58.97	62.90	18 $\frac{1}{2}$	18 $\frac{1}{2}$		
19	16.15	20.19	24.23	28.26	32.30	36.34	40.38	44.41	48.45	52.49	56.53	60.56	64.60	19	19		
19 $\frac{1}{2}$	16.57	20.72	24.86	29.00	33.15	37.30	41.44	45.58	49.72	53.87	58.02	62.15	66.30	19 $\frac{1}{2}$	19 $\frac{1}{2}$		
20	17.00	21.25	25.50	29.75	34.00	38.25	42.50	46.75	51.00	55.25	59.50	63.75	68.00	20	20		
20 $\frac{1}{2}$	17.42	21.78	26.14	30.49	34.85	39.21	43.56	47.92	52.27	56.63	60.99	65.34	69.70	20 $\frac{1}{2}$	20 $\frac{1}{2}$		
21	17.85	22.31	26.78	31.24	35.70	40.16	44.63	49.09	53.55	58.01	62.48	66.94	71.40	21	21		
21 $\frac{1}{2}$	18.27	22.84	27.41	31.98	36.55	41.12	45.69	50.26	54.82	59.39	63.97	68.53	73.10	21 $\frac{1}{2}$	21 $\frac{1}{2}$		
22	18.70	23.38	28.06	32.72	37.40	42.08	46.75	51.43	56.10	60.78	65.45	70.13	74.80	22	22		
22 $\frac{1}{2}$	19.12	23.91	28.69	33.46	38.25	43.04	47.81	52.58	57.37	62.16	66.94	71.72	76.50	22 $\frac{1}{2}$	22 $\frac{1}{2}$		
23	19.55	24.44	29.33	34.21	39.10	43.90	48.83	53.76	58.69	63.64	68.43	73.31	78.20	23	23		
23 $\frac{1}{2}$	19.97	24.97	29.96	34.95	39.95	44.95	49.94	54.93	59.92	64.92	69.91	74.90	79.90	23 $\frac{1}{2}$	23 $\frac{1}{2}$		
24	20.40	25.50	30.60	35.70	40.80	45.90	51.00	56.10	61.20	66.30	71.40	76.50	81.60	24	24		
24 $\frac{1}{2}$	20.82	26.08	31.24	36.44	41.65	46.86	52.06	57.27	62.47	67.68	72.89	78.09	83.30	24 $\frac{1}{2}$	24 $\frac{1}{2}$		
25	21.25	26.66	31.88	37.19	42.50	47.81	53.13	58.44	63.75	69.06	74.38	79.69	85.00	25	25		
25 $\frac{1}{2}$	21.67	27.09	32.51	37.93	43.35	48.77	54.19	59.61	65.02	70.44	75.86	81.28	86.70	25 $\frac{1}{2}$	25 $\frac{1}{2}$		
26	22.10	27.63	33.15	38.68	44.20	49.73	55.25	60.78	66.30	71.83	77.35	82.88	88.40	26	26		
26 $\frac{1}{2}$	22.52	28.16	33.79	39.42	45.05	50.69	56.31	61.94	67.57	73.21	78.84	84.47	90.10	26 $\frac{1}{2}$	26 $\frac{1}{2}$		
27	22.95	28.69	34.43	40.16	45.90	51.64	57.38	63.11	68.85	74.59	80.33	86.06	91.80	27	27		
27 $\frac{1}{2}$	23.37	29.22	35.06	40.90	46.75	52.60	58.44	64.28	70.12	75.97	81.81	87.66	93.50	27 $\frac{1}{2}$	27 $\frac{1}{2}$		
28	23.80	29.75	35.70	41.65	47.60	53.55	59.50	65.45	71.40	77.35	83.30	89.25	95.20	28	28		
28 $\frac{1}{2}$	24.22	30.28	36.34	42.39	48.45	54.51	60.56	66.62	72.67	78.73	84.79	90.84	96.90	28 $\frac{1}{2}$	28 $\frac{1}{2}$		
29	24.65	30.81	36.98	43.14	49.30	55.45	61.63	67.79	73.95	80.11	86.28	92.44	98.60	29	29		
29 $\frac{1}{2}$	25.07	31.34	37.61	43.88	50.15	56.42	62.69	68.96	75.22	81.49	87.76	94.03	100.3	29 $\frac{1}{2}$	29 $\frac{1}{2}$		
30	25.50	31.88	38.26	44.63	51.00	57.38	63.75	70.13	76.50	82.88	89.25	95.63	102.0	30	30		
30 $\frac{1}{2}$	25.92	32.41	38.89	45.37	51.85	58.34	64.81	71.30	77.77	84.26	90.74	97.22	103.7	30 $\frac{1}{2}$	30 $\frac{1}{2}$		
31	26.35	32.94	39.59	46.11	52.70	59.29	65.88	72.46	79.05	85.64	92.23	98.81	105.4	31	31		
31 $\frac{1}{2}$	26.77	33.47	40.16	46.86	53.55	60.25	66.94	73.63	80.32	87.02	93.71	100.4	107.1	31 $\frac{1}{2}$	31 $\frac{1}{2}$		
32	27.20	34.00	40.80	47.60	54.40	61.20	68.00	74.80	81.60	88.40	95.20	102.0	108.8	32	32		
32 $\frac{1}{2}$	27.62	34.53	41.44	48.34	55.25	62.10	69.06	75.97	82.87	89.78	96.69	103.6	110.5	32 $\frac{1}{2}$	32 $\frac{1}{2}$		
33	28.05	35.06	42.08	49.09	56.10	63.11	70.13	77.14	84.15	91.16	98.18	105.2	112.2	33	33		
33 $\frac{1}{2}$	28.47	35.59	42.71	49.88	56.95	64.07	71.19	78.31	85.42	92.54	99.66	106.8	113.9	33 $\frac{1}{2}$	33 $\frac{1}{2}$		
34	28.90	36.13	43.35	50.68	57.90	65.03	72.25	79.48	86.70	93.93	101.1	108.4	115.6	34	34		
34 $\frac{1}{2}$	29.32	36.66	43.99	51.32	58.65	65.99	73.31	80.65	87.97	95.31	102.6	110.0	117.3	34 $\frac{1}{2}$	34 $\frac{1}{2}$		
35	29.75	37.19	44.03	52.06	59.50	66.94	74.38	81.81	89.25	96.69	104.1	111.6	119.0	35	35		
35 $\frac{1}{2}$	30.17	37.72	44.56	52.80	60.35	67.90	75.44	82.98	90.52	98.07	105.6	113.1	120.7	35 $\frac{1}{2}$	35 $\frac{1}{2}$		
36	30.60	38.25	45.90	53.55	61.20	68.85	76.50	84.15	91.80	99.45	107.1	114.7	122.4	36	36		

Values for additional width of  $\frac{1}{4}$ "

$\frac{1}{4}$  218 266 319 372 425 478 531 584 638 691 744 797 850  $\frac{1}{4}$

# REDPATH, BROWN & CO., LIMITED.

## FLAT ROLLED STEEL

Weight per Lineal Foot in Lbs. (concluded).

Width in Inches.	THICKNESS IN FRACTIONS OF AN INCH.																Width in Inches.
	$\frac{1}{8}$	$\frac{1}{16}$	$\frac{3}{16}$	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{3}{8}$	$1\frac{1}{2}$	$1\frac{5}{8}$	
37	81.48	39.81	47.18	55.04	62.90	70.76	78.63	86.49	94.36	102.2	110.1	117.9	125.8				37
38	82.30	40.98	48.45	56.33	64.20	72.08	80.75	88.88	96.90	105.0	113.0	121.1	129.2				38
39	83.15	41.44	49.73	58.01	66.30	74.59	82.88	91.16	99.45	107.7	116.0	124.3	132.6				39
40	84.00	42.50	51.00	59.60	68.00	76.50	85.00	93.50	102.0	110.5	119.0	127.5	136.0				40
41	84.86	43.56	52.28	60.69	69.70	78.41	87.13	95.84	104.5	113.3	122.0	130.7	139.4				41
42	85.70	44.03	53.55	62.48	71.40	80.33	89.25	98.18	107.1	116.0	124.9	133.9	142.8				42
43	86.55	45.09	54.83	63.96	73.10	82.24	91.38	100.6	109.6	118.8	127.9	137.1	146.2				43
44	87.40	46.76	56.10	65.45	74.80	84.15	93.50	102.8	112.2	121.5	130.9	140.2	149.6				44
45	88.25	47.81	57.38	66.94	76.60	86.06	95.63	105.2	114.7	124.3	133.9	143.4	153.0				45
46	89.10	48.88	58.65	68.43	78.20	87.98	97.76	107.6	117.3	127.1	136.8	146.6	156.4				46
47	89.95	49.94	59.93	69.91	79.90	89.89	99.88	109.9	119.8	129.8	139.8	149.8	159.8				47
48	40.80	51.00	61.20	71.40	81.60	91.80	102.0	112.2	122.4	132.6	142.8	153.0	163.2				48
49	41.66	52.00	62.47	72.89	83.30	93.71	104.1	114.5	124.9	135.3	145.8	156.2	166.6				49
50	42.50	53.13	63.76	74.88	85.00	95.63	106.2	116.9	127.5	138.1	148.7	159.4	170.0				50
51	43.35	54.19	65.02	75.80	86.70	97.54	108.4	119.2	130.0	140.9	151.7	162.6	173.4				51
52	44.20	55.25	66.30	77.35	88.40	99.45	110.5	121.5	132.6	143.6	154.7	165.7	176.8				52
53	45.05	56.31	67.57	78.84	90.10	101.4	112.6	123.9	135.1	146.4	157.7	168.9	180.2				53
54	45.90	57.38	68.85	80.33	91.80	103.3	114.7	126.2	137.7	149.2	160.6	172.1	183.6				54
55	46.76	58.44	70.12	81.81	93.50	105.2	116.9	128.6	140.2	151.9	163.6	175.3	187.0				55
56	47.60	59.60	71.40	83.90	95.20	107.1	119.0	130.9	142.8	154.7	166.6	178.5	190.4				56
57	48.45	60.66	72.67	84.79	96.90	109.0	121.1	133.2	145.3	157.5	169.6	181.7	193.8				57
58	49.30	61.73	73.95	86.28	98.60	110.9	123.3	135.6	147.9	160.2	172.0	184.9	197.2				58
59	50.15	62.09	75.22	87.76	100.3	112.8	125.4	137.9	150.4	163.0	175.5	188.1	200.6				59
60	51.00	63.76	76.60	89.25	102.0	114.7	127.5	140.3	153.0	165.8	178.5	191.2	204.0				60
61	51.85	64.81	77.77	90.74	103.7	116.7	129.6	142.6	155.5	168.5	181.5	194.4	207.4				61
62	52.70	65.88	79.05	92.23	105.4	118.6	131.6	144.9	158.1	171.3	184.5	197.6	210.8				62
63	53.55	66.94	80.32	93.71	107.1	120.5	133.9	147.3	160.6	174.0	187.4	200.8	214.2				63
64	54.40	68.00	81.60	95.20	108.8	122.4	136.0	149.6	163.2	176.8	190.4	204.0	217.6				64
65	55.25	69.06	82.87	96.69	110.5	124.3	138.1	151.9	165.7	179.6	193.4	207.2	221.0				65
66	56.10	70.18	84.15	98.18	112.2	126.2	140.3	154.3	168.3	182.3	196.4	210.4	224.4				66
67	56.95	71.19	85.42	99.66	113.9	128.1	142.4	156.6	170.8	185.1	199.3	213.0	227.8				67
68	57.80	72.25	86.70	101.1	115.6	130.0	144.5	159.0	173.4	187.9	202.2	216.7	231.2				68
69	58.65	73.31	87.97	102.6	117.3	132.0	146.6	161.3	176.9	190.6	205.2	219.9	234.6				69
70	59.50	74.38	89.25	104.1	119.0	133.9	148.8	163.6	178.5	193.4	208.2	223.1	238.0				70
71	60.35	75.44	90.52	105.6	120.7	135.8	150.9	166.0	181.0	196.1	211.2	226.3	241.4				71
72	61.20	76.50	91.80	107.1	122.4	137.7	153.0	168.3	183.6	198.0	214.2	229.4	244.8				72
73	62.06	77.56	93.07	108.6	124.1	139.6	155.1	170.6	186.1	201.7	217.2	232.7	248.2				73
74	62.90	78.63	94.36	110.1	125.8	141.5	157.3	173.0	188.7	204.4	220.1	235.9	251.0				74
75	63.75	79.69	95.62	111.6	127.5	143.4	159.4	175.3	191.2	207.2	223.1	239.1	255.0				75
76	64.60	80.75	96.90	113.1	129.2	145.3	161.5	177.7	193.8	210.0	226.1	242.3	258.4				76
77	65.45	81.81	98.17	114.6	130.9	147.3	163.6	180.0	196.3	212.7	229.1	245.5	261.8				77
78	66.30	82.88	99.45	116.0	132.6	149.2	165.8	182.3	198.0	215.4	232.2	248.6	265.2				78
79	67.15	83.94	100.7	117.5	134.3	151.1	167.9	184.7	201.4	218.2	235.0	251.8	268.6				79
80	68.00	85.00	102.0	119.0	136.0	153.0	170.0	187.0	204.0	221.0	238.0	255.0	272.0				80

Values for additional widths of  $\frac{1}{8}$ ",  $\frac{1}{4}$ " and  $\frac{3}{8}$ "

$\frac{1}{8}$	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{3}{8}$	$1\frac{1}{2}$	$1\frac{5}{8}$	$1\frac{3}{4}$	$1\frac{7}{8}$	$2$	$2\frac{1}{8}$	$2\frac{1}{4}$	$2\frac{3}{8}$	$2\frac{1}{2}$	$2\frac{5}{8}$
213	260	319	372	425	478	531	584	638	691	744	797	850							
425	531	638	744	850	956	1063	1169	1276	1381	1488	1594	1700							
638	797	956	1110	1276	1434	1594	1753	1913	2072	2231	2391	2550							

# REDPATH, BROWN & CO., LIMITED.

## SQUARE AND ROUND STEEL

Weights in Lbs. per Lineal Foot and Areas in Square Inches.

Side or Diameter in inches.	Square. ■		Round. ●		Side or Diameter in inches.	Square. ■		Round. ●	
	Weight.	Area.	Weight.	Area.		Weight.	Area.	Weight.	Area.
$\frac{1}{8}$	1.20	.035	.094	.027	4	54.40	16.00	42.78	12.57
$\frac{1}{4}$	2.13	.062	.167	.049	4½	61.41	18.06	48.23	14.18
$\frac{3}{8}$	3.32	.097	.261	.076	4¾	68.85	20.25	54.07	15.90
$\frac{1}{2}$	4.78	.140	.376	.110	4½	76.71	22.56	60.25	17.73
$\frac{5}{8}$	6.51	.191	.511	.150	5	85.00	25.00	66.76	19.63
$\frac{3}{4}$	8.49	.250	.668	.196	5¼	93.71	27.56	73.60	21.65
$\frac{7}{8}$	10.76	.316	.845	.249	5½	102.85	30.25	80.78	23.76
$1$	1.328	.390	1.043	.306	5¾	112.41	33.06	88.20	25.97
$1\frac{1}{8}$	1.607	.472	1.262	.371	6	122.40	36.00	96.13	28.27
$1\frac{1}{4}$	1.912	.562	1.502	.442	6¼	132.81	39.06	104.81	30.68
$1\frac{3}{8}$	2.245	.660	1.763	.518	6½	143.65	42.25	112.82	33.18
$1\frac{1}{2}$	2.603	.765	2.044	.601	6¾	154.87	45.56	121.67	35.73
$1\frac{3}{4}$	2.988	.879	2.347	.690	7	166.60	49.00	130.85	38.48
$2$	3.400	1.000	2.670	.785	7¼	178.70	52.56	140.36	41.23
$2\frac{1}{8}$	4.303	1.265	3.330	.994	7½	191.25	56.25	150.21	44.18
$2\frac{1}{4}$	5.312	1.562	4.172	1.227	7¾	204.20	60.06	160.39	47.17
$2\frac{3}{8}$	6.428	1.890	5.049	1.485	8	217.60	64.00	170.90	50.26
$2\frac{1}{2}$	7.650	2.250	6.008	1.767	8¼	231.40	68.06	181.75	53.45
$2\frac{3}{4}$	8.978	2.640	7.051	2.074	8½	245.65	72.25	192.93	56.74
$3$	10.41	3.062	8.178	2.405	8¾	260.30	76.56	204.45	60.13
$3\frac{1}{8}$	11.95	3.515	9.388	2.761	9	275.40	81.00	216.36	63.62
$3\frac{1}{4}$	13.60	4.000	10.68	3.141	9¼	290.90	85.56	228.48	67.20
$3\frac{3}{8}$	15.35	4.515	12.06	3.546	9½	306.85	90.25	241.00	70.83
$3\frac{1}{2}$	17.21	5.062	13.52	3.976	9¾	323.20	95.06	253.85	74.66
$3\frac{3}{4}$	19.18	5.640	15.06	4.430	10	340.00	100.00	267.04	78.54
$4$	21.25	6.250	16.69	4.908	10¼	357.20	105.06	280.55	82.51
$4\frac{1}{8}$	23.43	6.890	18.40	5.412	10½	374.85	110.25	294.41	86.59
$4\frac{1}{4}$	25.71	7.562	20.19	5.939	10¾	392.90	115.56	308.59	90.76
$4\frac{3}{8}$	28.10	8.265	22.07	6.492	11	411.40	121.00	323.11	95.03
$4\frac{1}{2}$	30.60	9.000	24.03	7.068	11¼	440.05	132.25	353.15	103.87
$4\frac{3}{4}$	35.91	10.56	28.21	8.296	11½	449.05	133.25	353.15	103.87
$5$	41.65	12.25	32.71	9.621	12	489.60	144.00	384.53	113.09
$5\frac{1}{8}$	47.81	14.06	37.55	11.04					

# REDPATH, BROWN & CO., LIMITED.

## **MOMENTS OF INERTIA PER INCH WIDTH.**

Plates at various distances apart.

Distance between plates in inches.	THICKNESS OF PLATES IN INCHES.								
	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1	$1\frac{1}{8}$	$1\frac{1}{2}$	$1\frac{3}{4}$
8	13.16	18.08	23.29	28.78	31.57	40.67	47.07	53.80	60.86
9	16.49	22.58	28.99	35.72	42.77	50.17	57.90	65.99	74.44
10	20.19	27.58	35.32	43.41	51.85	60.67	69.86	79.43	89.39
11	24.27	33.08	42.27	51.81	61.81	72.17	82.93	94.11	105.7
12	28.72	39.08	49.85	61.03	72.63	84.67	97.14	110.1	123.4
14	38.75	52.58	66.88	81.66	96.92	113.7	128.9	145.7	163.0
15	44.33	60.08	76.33	93.09	110.4	128.2	146.5	165.4	184.8
16	50.28	68.08	86.41	105.3	124.7	144.7	165.2	186.8	208.0
18	63.32	85.58	108.4	131.9	156.0	180.7	206.0	231.9	258.5
20	77.85	105.1	133.0	161.5	190.8	220.7	251.3	282.6	314.5
24	111.4	150.1	189.5	229.8	279.8	312.7	355.3	398.8	443.1

Distance between plates in inches.	THICKNESS OF PLATES IN INCHES.								
	$1\frac{1}{8}$	$1\frac{1}{2}$	$1\frac{3}{4}$	$1\frac{7}{8}$	2	$2\frac{1}{8}$	$2\frac{1}{2}$	$2\frac{3}{8}$	$2\frac{1}{2}$
8	68.25	75.99	84.07	91.52	101.3	110.5	120.1	130.1	140.4
9	83.25	92.44	102.0	112.0	122.3	133.1	144.3	155.9	167.9
10	99.75	110.5	121.7	133.3	145.3	157.8	170.7	184.1	197.9
11	117.7	130.2	143.1	156.5	170.3	184.6	199.4	214.7	230.4
12	137.2	151.5	166.3	181.6	197.3	213.6	230.3	247.6	265.4
14	180.7	199.1	217.9	237.4	257.3	277.9	299.0	320.6	342.9
15	204.7	225.3	246.1	268.1	290.3	313.2	336.7	360.7	385.4
16	230.2	253.1	276.6	300.6	325.3	350.6	376.6	403.2	430.4
18	285.7	313.6	342.2	371.4	401.3	431.9	463.2	495.2	527.9
20	347.2	380.7	411.8	448.7	485.3	521.7	558.8	596.7	635.4
24	488.2	534.2	581.1	628.8	677.3	726.8	777.1	828.3	880.4

The above distances between plates correspond to the depths of the British Standard Sections of Joists and Channels.

## **NET MAXIMUM MOMENT OF INERTIA OF STEEL JOISTS.**

1 RIVET HOLE IN EACH FLANGE.

Size of Joist, - - -	$2\frac{1}{2} \times 7\frac{1}{2}$	$20 \times 7\frac{1}{2}$	$18 \times 7$	$16 \times 6$	$15 \times 6$	$15 \times 5$	$14 \times 6\frac{1}{2}$	$14 \times 6\frac{1}{2}$
Net Moment of Inertia, -	2398	1505	1025	648.2	550.0	374.3	472.9	391.7
Size of Joist, - - -	$12 \times 6\frac{1}{2}$	$12 \times 6\frac{1}{2}$	$12 \times 5$	$10 \times 6$	$10 \times 5$	$8 \times 6$	$8 \times 5$	
Net Moment of Inertia, -	331.9	279.2	191.1	186.5	125.9	97.65	76.56	

## **NET MAXIMUM MOMENT OF INERTIA OF STEEL CHANNELS.**

1 RIVET HOLE IN EACH FLANGE.

Size of Channel, - -	$15 \times 4$	$12 \times 3\frac{1}{2}$	$10 \times 3\frac{1}{2}$	$9 \times 3\frac{1}{2}$	$8 \times 3\frac{1}{2}$	$7 \times 3\frac{1}{2}$
Net Moment of Inertia, -	324.2	159.0	97.20	72.12	51.85	35.98

# REDPATH, BROWN & CO., LIMITED.

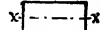
## MOMENTS OF INERTIA OF RECTANGLES.

Depth in inches. D.	WIDTH OF RECTANGLE IN INCHES = B.									Depth in inches. D.
	$\frac{1}{16}$	$\frac{1}{8}$	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1	
1	.005	.010	.021	.031	.042	.052	.062	.073	.083	1
2	.042	.083	.167	.250	.333	.417	.500	.583	.667	2
3	.141	.281	.562	.844	1.125	1.406	1.687	1.969	2.250	3
4	.333	.667	1.333	2.000	2.667	3.333	4.000	4.667	5.333	4
5	.651	1.302	2.604	3.906	5.208	6.510	7.812	9.115	10.42	5
6	1.125	2.250	4.500	6.750	9.000	11.25	13.50	15.75	18.00	6
7	1.786	3.573	7.146	10.72	14.29	17.86	21.44	25.01	28.58	7
8	2.667	5.333	10.67	16.00	21.33	26.67	32.00	37.33	42.67	8
9	3.797	7.594	15.19	22.78	30.37	37.97	45.56	53.16	60.75	9
10	5.208	10.42	20.83	31.25	41.67	52.08	62.50	72.92	83.33	10
11	6.932	13.86	27.73	41.59	55.46	69.32	83.19	97.05	110.9	11
12	9.000	18.00	36.00	54.00	72.00	90.00	108.00	126.0	144.0	12
13	11.44	22.89	45.77	68.66	91.54	114.4	137.3	160.2	183.1	13
14	14.29	28.58	57.17	85.75	114.3	142.9	171.5	200.1	228.7	14
15	17.58	35.16	70.31	105.5	140.6	175.8	210.9	246.1	281.2	15
16	21.33	42.67	85.33	128.0	170.7	213.3	256.0	298.7	341.3	16
17	25.69	51.38	102.8	153.5	201.7	255.9	307.1	358.2	409.4	17
18	30.37	60.75	121.5	182.2	243.0	303.7	364.5	425.2	486.0	18
19	35.72	71.45	142.9	214.3	285.8	357.2	428.7	500.1	571.6	19
20	41.67	83.33	166.7	250.0	333.3	416.7	500.0	583.3	666.7	20
21	48.23	96.47	192.9	289.4	385.9	482.3	578.8	675.3	771.7	21
22	55.46	110.9	221.8	332.7	443.7	554.6	665.5	776.4	887.3	22
23	63.37	126.7	253.5	380.2	507.0	633.7	760.4	887.2	1014	23
24	72.00	144.0	288.0	432.0	576.0	720.0	864.0	1008	1152	24
25	81.38	162.8	325.5	493.3	651.0	813.8	976.6	1139	1302	25
26	91.54	183.1	366.2	549.2	732.3	915.4	1098	1282	1465	26
27	102.5	205.0	410.1	615.1	820.1	1025	1230	1435	1640	27
28	114.3	228.7	457.3	686.0	914.7	1143	1372	1601	1829	28
29	127.0	254.1	508.1	762.2	1016	1270	1524	1778	2032	29
30	140.6	281.2	562.5	843.7	1125	1406	1687	1969	2250	30
31	155.2	310.3	620.6	931.0	1241	1552	1862	2172	2483	31
32	170.7	341.3	682.7	1024	1365	1707	2048	2389	2731	32
33	187.2	374.3	748.7	1123	1497	1872	2246	2620	2995	33
34	204.7	409.4	819.8	1228	1638	2047	2456	2866	3275	34
35	223.3	446.6	893.2	1340	1780	2233	2680	3126	3673	35
36	243.0	486.0	972.0	1458	1944	2430	2916	3402	3888	36
37	263.8	527.6	1055	1583	2111	2638	3166	3693	4221	37
38	285.6	571.6	1143	1715	2286	2856	3420	4001	4573	38
39	309.0	617.9	1236	1854	2472	3090	3707	4325	4943	39
40	333.3	666.7	1333	2000	2667	3333	4000	4667	5333	40
41	359.0	717.9	1436	2154	2872	3590	4308	5025	5743	41
42	385.9	771.7	1543	2315	3087	3859	4630	5402	6174	42

# REDPATH, BROWN & CO., LIMITED.



## MOMENTS OF INERTIA OF RECTANGLES.

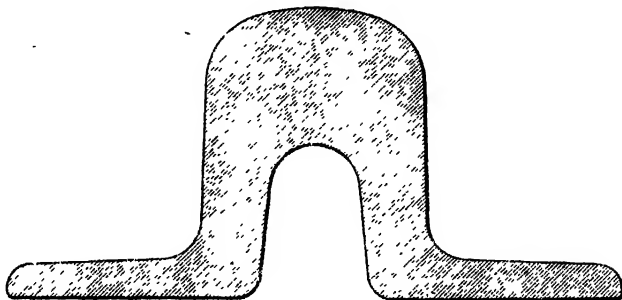


Depth in inches. D.	WIDTH OF RECTANGLE IN INCHES = b.									Depth in inches. D.
	1	2	3	4	5	6	7	8	9	
1	.004 .010	.009 .021	.013 .031	.018 .042	.022 .052	.026 .062	.031 .073	.035 .083	.040 .094	1
1 1/4	.020 .035 .056 .083	.041 .070 .112 .167	.061 .105 .167 .250	.081 .141 .223 .333	.102 .176 .279 .417	.122 .211 .335 .500	.142 .246 .391 .583	.163 .281 .447 .667	.183 .316 .502 .750	1 1/4
1 1/2	.119 .163	.237 .320	.356 .483	.475 .651	.593 .814	.712 .970	.831 1.139	.949 1.302	1.068 1.465	1 1/2
1 3/4	.217 .281	.433 .562	.650 .844	.867 1.125	1.083 1.406	1.300 1.687	1.517 1.969	1.733 2.250	1.950 2.531	1 3/4
2	.358 .447 .549 .667	.715 .893 1.099 1.333	1.073 1.340 1.643 2.000	1.430 1.786 2.197 2.667	1.738 2.233 2.747 3.333	2.145 2.679 3.236 4.000	2.503 3.116 3.545 4.667	2.861 3.573 4.395 5.333	3.218 4.019 4.944 6.000	2
2 1/4	.800 .949	1.599 1.898	2.399 2.818	3.199 3.797	3.998 4.716	4.798 5.605	5.598 6.615	6.397 7.594	7.197 8.543	2 1/4
2 1/2	1.116 1.302	2.233 2.604	3.349 3.906	4.465 5.208	5.582 6.510	6.698 7.812	7.814 9.115	8.931 10.417	10.017 11.719	2 1/2
Depth in inches. D.	WIDTH OF RECTANGLE IN INCHES = b.									Depth in inches. D.
	10	12	14	15	16	18	20	22	24	
1	.044 .104	.053 .125	.062 .146	.066 .156	.070 .167	.079 .187	.088 .208	.097 .229	.105 .250	1
1 1/4	.208 .262 .358 .433	.244 .422 .670 1.000	.285 .492 .782 1.167	.306 .627 .893 1.250	.326 .602 .893 1.333	.366 .633 1.005 1.500	.407 .703 1.117 1.667	.447 .774 1.223 1.833	.488 .844 1.240 2.000	1 1/4
1 1/2	1.187 1.627	1.424 1.953	1.661 2.278	1.780 2.441	1.908 2.604	2.136 2.929	2.373 3.255	2.611 3.580	2.848 3.906	1 1/2
1 3/4	2.167 2.812	2.600 3.375	3.033 3.937	3.250 4.219	3.466 4.500	3.900 5.062	4.333 5.625	4.767 6.187	5.199 6.750	1 3/4
2	3.576 4.466	4.271 5.359	5.006 6.252	5.364 6.699	5.721 7.140	6.436 8.038	7.152 8.932	7.867 9.825	8.582 10.719	2
2 1/4	5.493 6.667	6.592 8.000	7.391 9.333	8.240 10.000	8.789 10.667	9.888 12.000	10.986 13.333	12.086 14.667	13.184 16.000	2 1/4
2 1/2	7.997 9.493	9.590 11.391	11.195 13.289	11.995 14.239	12.794 15.187	14.394 17.086	15.993 18.984	17.593 20.883	19.191 22.781	2 1/2
2 3/4	11.163 13.021	13.396 15.625	15.629 18.229	16.745 19.531	17.862 20.833	20.094 23.437	22.327 26.042	24.559 28.646	26.793 31.250	2 3/4

REDPATH, BROWN & CO., LIMITED

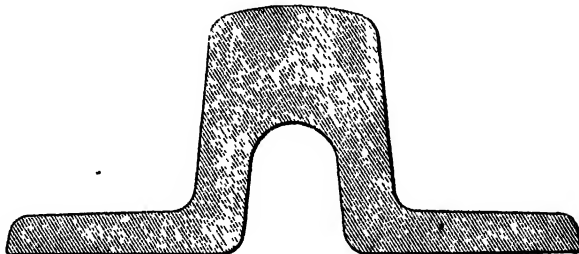
**STEEL BRIDGE RAILS.**

SCALE— $\frac{1}{2}$  FULL SIZE.



70 LBS. PER YARD.

SCALE— $\frac{1}{2}$  FULL SIZE.



56 LBS. PER YARD.

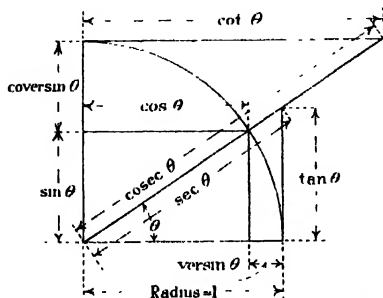
REDPATH, BROWN & CO., LIMITED.

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**MATHEMATICAL  
TABLES.**

**BRITISH AND METRIC  
WEIGHTS AND MEASURES,  
WITH  
CONVERSION FACTORS AND  
EQUIVALENTS.**

## TRIGONOMETRICAL EXPRESSIONS.



### Functions of $\pi$

Area of circle to radius 1	$\frac{1}{2} \pi$
Circumf. " " " diameter 1	$\pi$
Area " " " " 1	$\frac{\pi}{4}$
Volume " sphere " " 1	$\frac{\pi}{6}$
" " " " radius 1	$\frac{4\pi}{3}$

$$\pi = 3.1415926$$

$$\frac{\pi}{4} = 0.785398$$

$$\pi^2 = 9.869604$$

$$\pi^3 = 31.006276$$

$$\sqrt{\pi} = 1.772454$$

$$\sqrt[3]{\pi} = 1.464590$$

$$\frac{\pi}{6} = 0.523599$$

$$\frac{4\pi}{3} = 4.188790$$

The complement of an angle  $\theta = 90^\circ - \theta$ .

The supplement of an angle  $\theta = 180^\circ - \theta$ .

## TRIGONOMETRICAL EQUIVALENTS.

$$\sin \theta = \sqrt{1 - \cos^2 \theta}$$

$$\sin \theta = 1 \div \operatorname{cosec} \theta$$

$$\sin \theta = \cos \theta \div \cot \theta$$

$$\sin \theta = \tan \theta \div \sec \theta$$

$$\cos \theta = \sqrt{1 - \sin^2 \theta}$$

$$\cos \theta = 1 \div \sec \theta$$

$$\cos \theta = \sin \theta \times \cot \theta$$

$$\cos \theta = \sin \theta \div \tan \theta$$

$$\sec \theta = 1 \div \cos \theta$$

$$\sec \theta = \tan \theta \div \sin \theta$$

$$\operatorname{cosec} \theta = 1 \div \sin \theta$$

$$\tan \theta = 1 \div \cot \theta$$

$$\tan \theta = \sin \theta \div \cos \theta$$

$$\cot \theta = 1 \div \tan \theta$$

$$\cot \theta = \cos \theta \div \sin \theta$$

$$\operatorname{versin} \theta = 1 - \cos \theta$$

$$\operatorname{coversin} \theta = 1 - \sin \theta$$

$$1 = \tan \theta \times \cot \theta$$

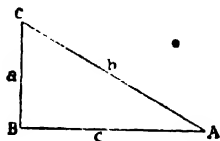
$$1 = \sin^2 \theta + \cos^2 \theta$$

$$\sec^2 \theta = 1 + \tan^2 \theta$$

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## TRIGONOMETRICAL FUNCTIONS.

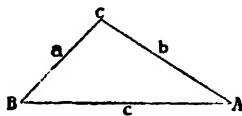
### Right-Angled Triangles.



$$\begin{aligned}\sin A &= \frac{a}{c} & \sec A &= \frac{c}{a} & \tan A &= \frac{a}{b} \\ \cos A &= \frac{b}{c} & \operatorname{cosec} A &= \frac{c}{b} & \cotan A &= \frac{b}{a} \\ \text{Versine } A &= \frac{b-c}{b} & \text{Coversine } A &= \frac{b-a}{b}\end{aligned}$$

Given.	Required.	Formulae.
a, b	A, C, c	$\sin A = \frac{a}{c}$ $\cos C = \frac{a}{b}$ $c = \sqrt{(b+a)(b-a)}$
a, c	A, C, b	$\tan A = \frac{a}{c}$ $\cot C = \frac{a}{c}$ $b = \sqrt{a^2 + c^2}$
A, a	C, c, b	$C = 90^\circ - A$ $c = a \times \cot A$ $b = \frac{a}{\sin A}$
A, b	C, a, c	$C = 90^\circ - A$ $a = b \times \sin A$ $c = b \times \cos A$
A, c	C, a, b	$C = 90^\circ - A$ $a = c \times \tan A$ $b = \frac{c}{\cos A}$

### Oblique-Angled Triangles.



Given.	Formulae.
A, B, C, a	$\left\{ \begin{aligned} &(a_2 \times \sin B \times \sin C) \div 2 \sin A \\ &\frac{1}{2} (c \times b \times \sin A) \\ &\sqrt{s(s-a)(s-b)(s-c)} \end{aligned} \right.$
A, b, c	
a, b, c	

Given.	Required.	Formulae.
A, C, a	c	$c = a \frac{\sin C}{\sin A}$
A, a, c	C	$\sin C = \frac{c \sin A}{a}$
a, c, B	A	$\tan A = \frac{a \sin B}{c - a \cos B}$
a, b, c	A	$\sin \frac{1}{2} A = \sqrt{\frac{(s-b)(s-c)}{b \times c}}$ ; $\cos \frac{1}{2} A = \sqrt{\frac{s(s-a)}{b \times c}}$ ; $\tan \frac{1}{2} A = \sqrt{\frac{(s-b)(s-c)}{s(s-a)}}$

# REDPATH, BROWN & CO., LIMITED.

## LOGARITHMS.

	0	1	2	3	4	5	6	7	8	*9	Mean Differences.								
											1	2	3	4	5	6	7	8	9
10	0000	0043	0086	0128	0170	0212	0253	0294	0334	0374	4	8	12	17	21	25	29	33	37
11	0414	0455	0492	0531	0569	0607	0645	0682	0719	0755	4	8	11	15	19	23	26	30	34
12	0792	0828	0864	0899	0934	0969	1004	1038	1072	1106	3	7	10	14	17	21	24	28	31
13	1139	1173	1206	1239	1271	1303	1335	1367	1399	1430	3	6	10	13	16	19	23	26	29
14	1461	1492	1523	1553	1584	1614	1644	1673	1703	1732	3	6	9	12	15	18	21	24	27
15	1761	1790	1818	1847	1875	1903	1931	1959	1987	2014	3	6	8	11	14	17	20	22	25
16	2041	2068	2095	2122	2148	2175	2201	2227	2253	2279	3	5	8	11	13	16	18	21	24
17	2304	2330	2355	2380	2405	2430	2455	2480	2504	2529	2	5	7	10	12	15	17	20	22
18	2553	2577	2601	2625	2648	2672	2695	2718	2742	2765	2	5	7	9	12	14	16	19	21
19	2788	2810	2833	2856	2878	2900	2923	2945	2967	2989	2	4	7	9	11	13	16	18	20
20	3010	3032	3054	3075	3096	3118	3139	3160	3181	3201	2	4	6	8	11	13	15	17	19
21	3222	3243	3263	3284	3304	3324	3345	3365	3385	3404	2	4	6	8	10	12	14	16	18
22	3424	3444	3464	3483	3502	3522	3541	3560	3579	3598	2	4	6	8	10	12	14	15	17
23	3617	3636	3655	3674	3692	3711	3729	3747	3766	3784	2	4	6	7	9	11	13	15	17
24	3802	3820	3838	3856	3874	3892	3909	3927	3945	3962	2	4	5	7	9	11	12	14	16
25	3979	3997	4014	4031	4048	4065	4082	4099	4116	4133	2	3	5	7	9	10	12	14	15
26	4150	4166	4183	4200	4216	4232	4249	4265	4281	4298	2	3	5	7	8	10	11	13	15
27	4314	4330	4346	4362	4378	4393	4409	4425	4440	4456	2	3	5	6	8	9	11	13	14
28	4472	4487	4502	4518	4533	4548	4564	4579	4594	4609	2	3	5	6	8	9	11	12	14
29	4624	4639	4654	4669	4683	4698	4713	4728	4742	4757	1	3	4	6	7	9	10	12	13
30	4771	4786	4800	4814	4829	4843	4857	4871	4886	4900	1	3	4	6	7	9	10	11	13
31	4914	4928	4942	4955	4969	4983	4997	5011	5024	5038	1	3	4	6	7	8	10	11	12
32	5051	5065	5079	5092	5105	5119	5132	5145	5159	5172	1	3	4	5	7	8	9	11	12
33	5185	5198	5211	5224	5237	5250	5263	5276	5289	5302	1	3	4	5	6	8	9	10	12
34	5315	5328	5340	5353	5366	5378	5391	5403	5416	5428	1	3	4	5	6	8	9	10	11
35	5441	5453	5465	5478	5490	5502	5514	5527	5539	5551	1	2	4	5	6	7	9	10	11
36	5563	5575	5587	5599	5611	5623	5635	5647	5658	5670	1	2	4	5	6	7	8	10	11
37	5682	5694	5705	5717	5729	5740	5752	5763	5775	5786	1	2	3	5	6	7	8	9	10
38	5798	5809	5821	5832	5843	5855	5866	5877	5888	5899	1	2	3	5	6	7	8	9	10
39	5911	5922	5933	5944	5955	5966	5977	5988	5999	6010	1	2	3	4	5	7	8	9	10
40	6021	6031	6042	6053	6064	6075	6085	6096	6107	6117	1	2	3	4	5	6	8	9	10
41	6128	6138	6149	6160	6170	6180	6191	6201	6212	6222	1	2	3	4	5	6	7	8	9
42	6232	6243	6253	6263	6274	6284	6294	6304	6314	6325	1	2	3	4	5	6	7	8	9
43	6335	6345	6355	6365	6375	6385	6395	6405	6415	6425	1	2	3	4	5	6	7	8	9
44	6435	6444	6454	6464	6474	6484	6493	6503	6513	6522	1	2	3	4	5	6	7	8	9
45	6532	6542	6551	6561	6571	6580	6590	6599	6609	6619	1	2	3	4	5	6	7	8	9
46	6628	6637	6646	6655	6665	6675	6684	6693	6702	6712	1	2	3	4	5	6	7	7	8
47	6721	6730	6739	6749	6758	6767	6776	6785	6794	6803	1	2	3	4	5	6	7	7	8
48	6812	6821	6830	6839	6848	6857	6866	6875	6884	6893	1	2	3	4	4	5	6	6	7
49	6902	6911	6920	6928	6937	6946	6955	6964	6972	6981	1	2	3	4	4	5	6	7	8
50	6990	6999	7007	7016	7024	7033	7042	7050	7059	7067	1	2	3	3	4	5	6	7	8
51	7076	7084	7093	7101	7110	7118	7126	7135	7143	7152	1	2	3	3	4	5	6	7	8
52	7160	7168	7177	7185	7193	7202	7210	7218	7226	7235	1	2	3	3	4	5	6	7	7
53	7243	7251	7259	7267	7275	7284	7292	7300	7308	7316	1	2	2	3	4	5	6	6	7
54	7324	7332	7340	7348	7356	7364	7372	7380	7388	7396	1	2	2	3	4	5	6	6	7

# REDPATH, BROWN & CO., LIMITED.

## LOGARITHMS.

	0	1	2	3	4	5	6	7	8	9	Mean Differences.								
											1	2	3	4	5	6	7	8	9
55	7404	7412	7419	7427	7435	7443	7451	7459	7466	7474	1	2	2	3	4	5	5	6	7
56	7482	7490	7497	7505	7513	7520	7528	7536	7543	7551	1	2	2	3	4	5	5	6	7
57	7559	7566	7574	7582	7589	7597	7604	7612	7619	7627	1	2	2	3	4	5	5	6	7
58	7634	7642	7649	7657	7664	7672	7679	7686	7694	7701	1	1	2	3	4	4	5	6	7
59	7709	7716	7723	7731	7738	7745	7752	7760	7767	7774	1	1	2	3	4	4	5	6	7
60	7782	7789	7796	7803	7810	7818	7825	7832	7839	7846	1	1	2	3	4	4	5	6	6
61	7853	7860	7868	7875	7882	7889	7896	7903	7910	7917	1	1	2	3	4	4	5	6	6
62	7924	7931	7938	7945	7952	7959	7966	7973	7980	7987	1	1	2	3	4	4	5	6	6
63	7993	8000	8007	8014	8021	8028	8035	8041	8048	8055	1	1	2	3	4	4	5	6	6
64	8062	8069	8075	8082	8089	8096	8102	8109	8116	8122	1	1	2	3	4	4	5	6	6
65	8129	8136	8142	8149	8156	8162	8169	8176	8182	8189	1	1	2	3	4	4	5	6	6
66	8195	8202	8209	8215	8222	8228	8235	8241	8248	8254	1	1	2	3	4	4	5	6	6
67	8261	8267	8274	8280	8287	8293	8299	8306	8312	8319	1	1	2	3	4	4	5	6	6
68	8325	8331	8338	8344	8351	8357	8363	8370	8376	8382	1	1	2	3	4	4	5	6	6
69	8388	8395	8401	8407	8414	8420	8426	8432	8439	8445	1	1	2	3	4	4	5	6	6
70	8451	8457	8463	8470	8476	8482	8488	8494	8500	8506	1	1	2	3	4	4	5	6	6
71	8513	8519	8525	8531	8537	8543	8549	8555	8561	8567	1	1	2	3	4	4	5	6	6
72	8573	8579	8585	8591	8597	8603	8609	8615	8621	8627	1	1	2	3	4	4	5	6	6
73	8633	8639	8645	8651	8657	8663	8669	8675	8681	8687	1	1	2	3	4	4	5	6	6
74	8692	8698	8704	8710	8716	8722	8727	8733	8739	8745	1	1	2	3	4	4	5	6	6
75	8751	8756	8762	8768	8774	8779	8785	8791	8797	8802	1	1	2	3	4	4	5	6	6
76	8808	8814	8820	8825	8831	8837	8842	8848	8854	8859	1	1	2	3	4	4	5	6	6
77	8865	8871	8876	8882	8887	8893	8899	8904	8910	8915	1	1	2	3	4	4	5	6	6
78	8921	8927	8932	8938	8943	8949	8954	8960	8965	8971	1	1	2	3	4	4	5	6	6
79	8976	8982	8987	8993	8998	9004	9009	9015	9020	9025	1	1	2	3	4	4	5	6	6
80	9031	9036	9042	9047	9053	9058	9063	9069	9074	9079	1	1	2	3	4	4	5	6	6
81	9085	9090	9096	9101	9106	9112	9117	9122	9128	9133	1	1	2	3	4	4	5	6	6
82	9138	9143	9149	9154	9159	9165	9170	9175	9180	9186	1	1	2	3	4	4	5	6	6
83	9191	9196	9201	9206	9212	9217	9222	9227	9232	9238	1	1	2	3	4	4	5	6	6
84	9243	9248	9253	9258	9263	9269	9274	9279	9284	9289	1	1	2	3	4	4	5	6	6
85	9294	9299	9304	9309	9315	9320	9325	9330	9335	9340	1	1	2	3	4	4	5	6	6
86	9345	9350	9355	9360	9365	9370	9375	9380	9385	9390	1	1	2	3	4	4	5	6	6
87	9395	9400	9405	9410	9415	9420	9425	9430	9435	9440	0	1	1	2	3	3	4	4	5
88	9445	9450	9455	9460	9465	9470	9474	9479	9484	9489	0	1	1	2	3	3	4	4	5
89	9494	9499	9504	9509	9513	9518	9523	9528	9533	9538	0	1	1	2	3	3	4	4	5
90	9542	9547	9552	9557	9562	9566	9571	9576	9581	9586	0	1	1	2	3	3	4	4	5
91	9590	9595	9600	9605	9609	9614	9619	9624	9628	9633	0	1	1	2	3	3	4	4	5
92	9638	9643	9647	9652	9657	9661	9666	9671	9675	9680	0	1	1	2	3	3	4	4	5
93	9685	9689	9694	9699	9703	9708	9713	9717	9722	9727	0	1	1	2	3	3	4	4	5
94	9731	9736	9741	9745	9750	9754	9759	9763	9768	9773	0	1	1	2	3	3	4	4	5
95	9777	9782	9786	9791	9795	9800	9805	9809	9814	9818	0	1	1	2	3	3	4	4	5
96	9823	9827	9832	9836	9841	9845	9850	9854	9859	9863	0	1	1	2	3	3	4	4	5
97	9868	9872	9877	9881	9886	9890	9894	9899	9903	9908	0	1	1	2	3	3	4	4	5
98	9912	9917	9921	9926	9930	9934	9939	9943	9948	9952	0	1	1	2	3	3	4	4	5
99	9956	9961	9965	9969	9974	9978	9983	9987	9991	9996	0	1	1	2	3	3	4	4	5

# REDPATH, BROWN & CO., LIMITED.

## ANTILOGARITHMS.

	0	1	2	3	4	5	6	7	8	9	Mean Differences.								
											1	2	3	4	5	6	7	8	9
*00	1000	1002	1005	1007	1009	1012	1014	1016	1019	1021	0	0	1	1	1	1	2	2	2
*01	1023	1026	1028	1030	1033	1035	1038	1040	1042	1045	0	0	1	1	1	1	2	2	2
*02	1047	1050	1052	1054	1057	1059	1062	1064	1067	1069	0	0	1	1	1	1	2	2	2
*03	1072	1074	1076	1079	1081	1084	1086	1089	1091	1094	0	0	1	1	1	1	2	2	2
*04	1096	1099	1102	1104	1107	1109	1112	1114	1117	1119	0	0	1	1	1	1	2	2	2
*05	1122	1125	1127	1130	1132	1135	1138	1140	1143	1146	0	1	1	1	1	2	2	2	2
*06	1148	1151	1153	1156	1159	1161	1164	1167	1169	1172	0	1	1	1	1	2	2	2	2
*07	1175	1178	1180	1183	1186	1189	1191	1194	1197	1199	0	1	1	1	1	2	2	2	2
*08	1202	1205	1208	1211	1213	1216	1219	1222	1225	1227	0	1	1	1	1	2	2	2	2
*09	1230	1233	1236	1239	1242	1245	1247	1250	1253	1256	0	1	1	1	1	2	2	2	2
*10	1259	1262	1265	1268	1271	1274	1276	1279	1282	1285	0	1	1	1	1	2	2	2	2
*11	1288	1291	1294	1297	1300	1303	1306	1309	1312	1315	0	1	1	1	1	2	2	2	2
*12	1318	1321	1324	1327	1330	1334	1337	1340	1343	1346	0	1	1	1	1	2	2	2	2
*13	1349	1352	1355	1358	1361	1365	1368	1371	1374	1377	0	1	1	1	1	2	2	2	2
*14	1380	1384	1387	1390	1393	1396	1400	1403	1406	1409	0	1	1	1	1	2	2	2	2
*15	1413	1416	1419	1422	1426	1429	1433	1435	1438	1441	0	1	1	1	1	2	2	2	2
*16	1445	1449	1452	1455	1459	1462	1466	1469	1473	1476	0	1	1	1	1	2	2	2	2
*17	1479	1483	1486	1489	1493	1496	1500	1503	1507	1510	0	1	1	1	1	2	2	2	2
*18	1514	1517	1521	1524	1528	1531	1535	1538	1542	1545	0	1	1	1	1	2	2	2	2
*19	1549	1552	1556	1560	1563	1567	1570	1574	1578	1581	0	1	1	1	1	2	2	2	2
*20	1585	1589	1592	1596	1600	1603	1607	1611	1614	1618	0	1	1	1	1	2	2	2	2
*21	1622	1626	1629	1633	1637	1641	1645	1648	1652	1656	0	1	1	1	1	2	2	2	2
*22	1660	1663	1667	1671	1675	1679	1683	1687	1690	1694	0	1	1	1	1	2	2	2	2
*23	1698	1702	1706	1710	1714	1718	1722	1726	1730	1734	0	1	1	1	1	2	2	2	2
*24	1738	1742	1746	1750	1754	1758	1762	1766	1770	1774	0	1	1	1	1	2	2	2	2
*25	1778	1782	1786	1791	1795	1799	1803	1807	1811	1815	0	1	1	1	1	2	2	2	2
*26	1820	1824	1828	1832	1837	1841	1845	1849	1854	1858	0	1	1	1	1	2	2	2	2
*27	1862	1866	1871	1875	1879	1884	1888	1892	1897	1901	0	1	1	1	1	2	2	2	2
*28	1905	1910	1914	1919	1923	1928	1932	1936	1941	1945	0	1	1	1	1	2	2	2	2
*29	1950	1954	1959	1963	1968	1972	1977	1982	1986	1991	0	1	1	1	1	2	2	2	2
*30	1995	2000	2004	2009	2014	2018	2023	2028	2032	2037	0	1	1	1	1	2	2	2	2
*31	2042	2046	2051	2056	2061	2065	2070	2075	2080	2084	0	1	1	1	1	2	2	2	2
*32	2089	2094	2099	2104	2109	2113	2118	2123	2128	2133	0	1	1	1	1	2	2	2	2
*33	2138	2143	2148	2153	2158	2163	2168	2173	2178	2183	0	1	1	1	1	2	2	2	2
*34	2188	2193	2198	2203	2208	2213	2218	2223	2228	2234	1	1	2	2	2	3	3	3	3
*35	2239	2244	2249	2254	2259	2265	2270	2275	2280	2286	1	1	2	2	2	3	3	3	3
*36	2291	2296	2301	2307	2312	2317	2323	2328	2333	2339	1	1	2	2	2	3	3	3	3
*37	2344	2350	2355	2360	2366	2371	2377	2382	2388	2393	1	1	2	2	2	3	3	3	3
*38	2399	2404	2410	2415	2421	2427	2432	2437	2443	2449	1	1	2	2	2	3	3	3	3
*39	2455	2460	2466	2472	2477	2483	2489	2495	2500	2506	1	1	2	2	2	3	3	3	3
*40	2512	2518	2523	2529	2535	2541	2547	2553	2559	2564	1	1	2	2	2	3	3	3	3
*41	2570	2576	2582	2588	2594	2600	2606	2612	2618	2624	1	1	2	2	2	3	3	3	3
*42	2630	2636	2642	2649	2655	2661	2667	2673	2679	2685	1	1	2	2	2	3	3	3	3
*43	2692	2698	2704	2710	2716	2723	2729	2735	2742	2748	1	1	2	2	2	3	3	3	3
*44	2754	2761	2767	2773	2780	2786	2793	2799	2805	2812	1	1	2	2	2	3	3	3	3
*45	2818	2825	2831	2838	2844	2851	2858	2864	2871	2877	1	1	2	2	2	3	3	3	3
*46	2884	2891	2897	2904	2911	2917	2924	2931	2938	2944	1	1	2	2	2	3	3	3	3
*47	2951	2958	2965	2972	2979	2985	2992	2999	3006	3013	1	1	2	2	2	3	3	3	3
*48	3020	3027	3034	3041	3048	3055	3062	3069	3076	3083	1	1	2	2	2	3	3	3	3
*49	3090	3097	3105	3112	3119	3126	3133	3141	3148	3155	1	1	2	2	2	3	3	3	3

# REDPATH, BROWN & CO., LIMITED.

## ANTILOGARITHMS.

											Mean Differences.								
	0	1	2	3	4	5	6	7	8	9	1	2	3	4	5	6	7	8	9
60	8162	3170	3177	8184	3192	8199	3206	3214	3221	3228	1	1	2	3	4	5	6	7	
61	8236	3243	3251	3258	3266	8273	3281	3289	3297	3304	1	2	2	3	4	5	6	7	
62	3311	3319	3327	8334	3342	3350	3357	3365	3373	3381	1	2	2	3	4	5	6	7	
63	8388	3396	3404	3412	3420	3428	3436	3443	3451	3459	1	2	2	3	4	5	6	7	
64	3467	3475	3483	3491	3499	3507	3514	3522	3530	3538	1	2	2	3	4	5	6	7	
65	3545	3553	3561	3569	3577	3585	3593	3601	3609	3617	1	2	2	3	4	5	6	7	
66	3625	3633	3641	3649	3657	3665	3673	3681	3689	3697	1	2	2	3	4	5	6	7	
67	3705	3713	3721	3729	3737	3745	3753	3761	3769	3777	1	2	2	3	4	5	6	7	
68	3785	3793	3801	3809	3817	3825	3833	3841	3849	3857	1	2	2	3	4	5	6	7	
69	3865	3873	3881	3889	3897	3905	3913	3921	3929	3937	1	2	2	3	4	5	6	7	
70	3945	3953	3961	3969	3977	3985	3993	4001	4009	4017	1	2	2	3	4	5	6	7	
71	4025	4033	4041	4049	4057	4065	4073	4081	4089	4097	1	2	2	3	4	5	6	7	
72	4105	4113	4121	4129	4137	4145	4153	4161	4169	4177	1	2	2	3	4	5	6	7	
73	4185	4193	4201	4209	4217	4225	4233	4241	4249	4257	1	2	2	3	4	5	6	7	
74	4265	4273	4281	4289	4297	4305	4313	4321	4329	4337	1	2	2	3	4	5	6	7	
75	4345	4353	4361	4369	4377	4385	4393	4401	4409	4417	1	2	2	3	4	5	6	7	
76	4425	4433	4441	4449	4457	4465	4473	4481	4489	4497	1	2	2	3	4	5	6	7	
77	4505	4513	4521	4529	4537	4545	4553	4561	4569	4577	1	2	2	3	4	5	6	7	
78	4585	4593	4601	4609	4617	4625	4633	4641	4649	4657	1	2	2	3	4	5	6	7	
79	4665	4673	4681	4689	4697	4705	4713	4721	4729	4737	1	2	2	3	4	5	6	7	
80	4745	4753	4761	4769	4777	4785	4793	4801	4809	4817	1	2	2	3	4	5	6	7	
81	4825	4833	4841	4849	4857	4865	4873	4881	4889	4897	1	2	2	3	4	5	6	7	
82	4905	4913	4921	4929	4937	4945	4953	4961	4969	4977	1	2	2	3	4	5	6	7	
83	4985	4993	5001	5009	5017	5025	5033	5041	5049	5057	1	2	2	3	4	5	6	7	
84	5065	5073	5081	5089	5097	5105	5113	5121	5129	5137	1	2	2	3	4	5	6	7	
85	5145	5153	5161	5169	5177	5185	5193	5201	5209	5217	1	2	2	3	4	5	6	7	
86	5225	5233	5241	5249	5257	5265	5273	5281	5289	5297	1	2	2	3	4	5	6	7	
87	5305	5313	5321	5329	5337	5345	5353	5361	5369	5377	1	2	2	3	4	5	6	7	
88	5385	5393	5401	5409	5417	5425	5433	5441	5449	5457	1	2	2	3	4	5	6	7	
89	5465	5473	5481	5489	5497	5505	5513	5521	5529	5537	1	2	2	3	4	5	6	7	
90	5545	5553	5561	5569	5577	5585	5593	5601	5609	5617	1	2	2	3	4	5	6	7	
91	5625	5633	5641	5649	5657	5665	5673	5681	5689	5697	1	2	2	3	4	5	6	7	
92	5705	5713	5721	5729	5737	5745	5753	5761	5769	5777	1	2	2	3	4	5	6	7	
93	5785	5793	5801	5809	5817	5825	5833	5841	5849	5857	1	2	2	3	4	5	6	7	
94	5865	5873	5881	5889	5897	5905	5913	5921	5929	5937	1	2	2	3	4	5	6	7	
95	5945	5953	5961	5969	5977	5985	5993	6001	6009	6017	1	2	2	3	4	5	6	7	
96	6025	6033	6041	6049	6057	6065	6073	6081	6089	6097	1	2	2	3	4	5	6	7	
97	6105	6113	6121	6129	6137	6145	6153	6161	6169	6177	1	2	2	3	4	5	6	7	
98	6185	6193	6201	6209	6217	6225	6233	6241	6249	6257	1	2	2	3	4	5	6	7	
99	6265	6273	6281	6289	6297	6305	6313	6321	6329	6337	1	2	2	3	4	5	6	7	

# REDPATH, BROWN & CO., LIMITED.

## NATURAL SINES.

	0'	6'	12'	18'	24'	30'	36'	42'	48'	54'	Mean Differences.				
											1'	2'	3'	4'	5'
0°	0000	6017	0035	0052	0070	0087	0105	0122	0140	0157	3	5	9	12	15
1	0175	0192	0209	0227	0244	0262	0279	0297	0314	0332	3	6	9	12	15
2	0349	0366	0384	0401	0419	0436	0454	0471	0488	0506	3	6	9	12	15
3	0523	0541	0558	0576	0593	0610	0628	0645	0663	0680	3	6	9	12	15
4	0698	0715	0732	0750	0767	0785	0802	0819	0837	0854	3	6	9	12	14
5	0872	0889	0906	0924	0941	0958	0975	0993	1011	1028	3	6	9	12	14
6	1045	1063	1080	1097	1115	1132	1149	1167	1184	1201	3	6	9	12	14
7	1219	1236	1253	1271	1288	1305	1323	1340	1357	1374	3	6	9	12	14
8	1392	1409	1426	1444	1461	1478	1495	1513	1530	1547	3	6	9	12	14
9	1564	1582	1599	1616	1633	1650	1668	1685	1702	1719	3	6	9	12	14
10	1736	1754	1771	1788	1805	1822	1840	1857	1874	1891	3	6	9	12	14
11	1908	1925	1942	1959	1977	1994	2011	2028	2045	2062	3	6	9	11	13
12	2079	2096	2113	2130	2147	2164	2181	2198	2215	2233	3	6	9	11	14
13	2250	2267	2284	2300	2317	2334	2351	2368	2385	2402	3	6	8	11	13
14	2419	2436	2453	2470	2487	2504	2521	2538	2554	2571	3	6	8	11	14
15	2588	2605	2622	2639	2656	2672	2689	2706	2723	2740	3	6	8	11	14
16	2756	2773	2790	2807	2823	2840	2857	2874	2890	2907	3	6	8	11	14
17	2924	2940	2957	2974	2990	3007	3024	3040	3057	3074	3	6	8	11	14
18	3090	3107	3123	3140	3156	3173	3190	3206	3223	3239	3	6	8	11	14
19	3256	3272	3289	3305	3322	3338	3355	3371	3387	3404	3	5	8	11	14
20	3420	3437	3453	3469	3486	3502	3518	3535	3551	3567	3	5	8	11	14
21	3584	3600	3616	3633	3649	3665	3681	3697	3714	3730	3	5	8	11	14
22	3746	3762	3778	3795	3811	3827	3843	3859	3875	3891	3	5	8	11	14
23	3907	3923	3939	3955	3971	3987	4003	4019	4035	4051	3	5	8	11	14
24	4067	4083	4099	4115	4131	4147	4163	4179	4195	4210	3	5	8	11	13
25	4226	4242	4258	4274	4289	4305	4321	4337	4352	4368	3	5	8	11	13
26	4384	4399	4415	4431	4446	4462	4478	4493	4509	4524	3	5	8	10	13
27	4540	4555	4571	4586	4602	4617	4633	4648	4664	4679	3	5	8	10	13
28	4695	4710	4726	4741	4756	4772	4787	4802	4818	4833	3	5	8	10	13
29	4848	4863	4879	4894	4909	4924	4939	4955	4970	4985	3	5	8	10	13
30	5000	5015	5030	5045	5060	5075	5090	5105	5120	5135	3	5	8	10	13
31	5150	5165	5180	5195	5210	5225	5240	5255	5270	5284	2	5	7	10	12
32	5299	5314	5329	5344	5358	5373	5388	5402	5417	5432	2	5	7	10	12
33	5446	5461	5476	5490	5505	5519	5534	5548	5563	5577	2	5	7	10	12
34	5592	5606	5621	5635	5650	5664	5678	5693	5707	5721	2	5	7	10	12
35	5736	5750	5764	5779	5793	5807	5821	5835	5850	5864	2	5	7	10	12
36	5878	5892	5906	5920	5934	5948	5962	5976	5990	6004	2	5	7	9	12
37	6018	6032	6046	6060	6074	6088	6101	6115	6129	6143	2	5	7	9	12
38	6157	6170	6184	6198	6211	6225	6239	6252	6266	6280	2	5	7	9	11
39	6293	6307	6320	6334	6347	6361	6374	6388	6401	6414	2	4	7	9	11
40	6428	6441	6455	6468	6481	6494	6508	6521	6534	6547	2	4	7	9	11
41	6561	6574	6587	6600	6613	6626	6639	6652	6665	6678	2	4	7	9	11
42	6691	6704	6717	6730	6743	6756	6769	6782	6794	6807	2	4	6	9	11
43	6820	6833	6845	6858	6871	6884	6896	6909	6921	6934	2	4	6	8	11
44	6947	6959	6972	6984	6997	7009	7022	7034	7046	7059	2	4	6	8	10

# REDPATH, BROWN & CO., LIMITED.

## NATURAL SINES.

	0'	6'	12'	18'	24'	30'	36'	42'	48'	54'	Mean Differences.				
											1'	2'	3'	4'	5'
45	7071	7083	7096	7108	7120	7133	7145	7157	7169	7181	2	4	6	8	10
46	7198	7206	7218	7230	7242	7254	7266	7278	7290	7302	2	4	6	8	10
47	7314	7325	7337	7349	7361	7373	7385	7396	7408	7420	2	4	6	8	10
48	7431	7443	7455	7466	7478	7490	7501	7513	7524	7536	2	4	6	8	10
49	7547	7559	7570	7581	7593	7604	7615	7627	7638	7649	2	4	6	8	9
50	7680	7672	7683	7694	7705	7716	7727	7738	7749	7760	2	4	6	7	9
51	7771	7782	7793	7804	7815	7826	7837	7848	7859	7869	2	4	5	7	9
52	7880	7891	7902	7912	7923	7934	7944	7955	7965	7976	2	4	5	7	9
53	7986	7997	8007	8018	8028	8039	8049	8059	8070	8080	2	3	5	7	9
54	8090	8100	8111	8121	8131	8141	8151	8161	8171	8181	2	3	5	7	8
55	8192	8202	8211	8221	8231	8241	8251	8261	8271	8281	2	3	5	7	8
56	8290	8300	8310	8320	8330	8340	8350	8360	8370	8381	2	3	5	6	8
57	8397	8406	8416	8425	8434	8443	8453	8462	8471	8481	2	3	5	6	8
58	8490	8499	8508	8517	8526	8535	8545	8554	8563	8572	2	3	5	6	8
59	8572	8581	8590	8599	8607	8616	8625	8634	8643	8652	1	3	4	6	7
60	8660	8669	8678	8686	8695	8704	8712	8721	8729	8738	1	3	4	6	7
61	8746	8755	8763	8771	8780	8788	8796	8805	8813	8821	1	3	4	6	7
62	8830	8838	8846	8854	8862	8870	8878	8886	8894	8902	1	2	4	5	7
63	8910	8918	8926	8934	8942	8949	8957	8965	8973	8980	1	3	4	5	6
64	8988	8996	9003	9011	9018	9026	9033	9041	9048	9056	1	3	4	5	6
65	9063	9070	9078	9085	9092	9100	9107	9114	9121	9128	1	2	4	5	6
66	9135	9143	9150	9157	9164	9171	9178	9184	9191	9198	1	2	3	5	6
67	9206	9212	9219	9225	9232	9239	9245	9252	9259	9265	1	2	3	4	6
68	9272	9278	9285	9291	9298	9304	9311	9317	9323	9330	1	2	3	4	5
69	9336	9342	9348	9354	9361	9367	9373	9379	9385	9391	1	2	3	4	5
70	9397	9403	9409	9415	9421	9426	9432	9438	9444	9449	1	2	3	4	5
71	9455	9461	9466	9472	9478	9483	9489	9494	9500	9505	1	2	3	4	5
72	9511	9516	9521	9527	9532	9537	9542	9548	9553	9558	1	2	3	4	4
73	9563	9568	9573	9578	9583	9588	9593	9598	9603	9608	1	2	2	3	4
74	9613	9617	9622	9627	9632	9636	9641	9646	9650	9655	1	2	2	3	4
75	9669	9664	9668	9673	9677	9681	9686	9690	9694	9699	1	1	2	3	4
76	9703	9707	9711	9716	9720	9724	9728	9732	9736	9740	1	1	2	3	3
77	9744	9748	9751	9755	9759	9763	9767	9770	9774	9778	1	1	2	3	3
78	9781	9785	9789	9792	9796	9799	9803	9806	9810	9813	1	1	2	2	3
79	9816	9820	9823	9826	9829	9833	9836	9839	9842	9845	1	1	2	2	3
80	9848	9851	9854	9857	9860	9863	9866	9869	9871	9874	0	1	1	2	2
81	9877	9880	9882	9885	9888	9890	9893	9895	9898	9900	0	1	1	2	2
82	9903	9905	9907	9910	9912	9914	9917	9919	9921	9923	0	1	1	2	2
83	9925	9928	9930	9933	9934	9936	9938	9940	9942	9943	0	1	1	1	2
84	9945	9947	9949	9951	9952	9954	9956	9957	9959	9960	0	1	1	1	1
85	9962	9963	9965	9966	9968	9969	9971	9972	9973	9974	0	0	1	1	1
86	9976	9977	9978	9979	9980	9981	9982	9983	9984	9985	0	0	1	1	1
87	9986	9987	9988	9989	9990	9990	9991	9992	9993	9994	0	0	0	1	1
88	9994	9995	9995	9996	9996	9997	9997	9997	9998	9998	0	0	0	0	0
89	9998	9999	9999	9999	9999	1'000	1'000	1'000	1'000	1'000	0	0	0	0	0

# REDPATH, BROWN & CO., LIMITED.

## NATURAL COSINES.

N.B.—Subtract Mean Differences.

	0'	6'	12'	18'	24'	30'	36'	42'	48'	54'	Mean Differences.				
											1'	2'	3'	4'	5'
0°	1'000	1'000	1'000	1'000	1'000	1'000	9999	9999	9999	9999	0	0	0	0	0
1	9998	9998	9998	9997	9997	9997	9996	9996	9995	9995	0	0	0	0	0
2	9994	9993	9993	9992	9991	9990	9990	9989	9988	9987	0	0	0	1	1
3	9986	9985	9984	9983	9982	9981	9980	9979	9978	9977	0	0	1	1	1
4	9976	9974	9973	9972	9971	9969	9968	9966	9965	9963	0	0	1	1	1
5	9902	9900	9899	9907	9896	9954	9952	9951	9949	9947	0	1	1	1	2
6	9845	9843	9842	9840	9838	9836	9834	9832	9830	9828	0	1	1	1	2
7	9825	9823	9821	9819	9817	9814	9812	9810	9807	9805	0	1	1	2	2
8	9803	9800	9808	9805	9803	9800	9888	9885	9882	9880	0	1	1	2	2
9	9877	9874	9871	9869	9866	9863	9860	9857	9854	9851	0	1	1	2	2
10	9848	9845	9842	9839	9836	9833	9829	9826	9823	9820	1	1	2	2	3
11	9816	9813	9810	9806	9803	9799	9796	9792	9789	9785	1	1	2	2	3
12	9781	9778	9774	9770	9767	9763	9759	9755	9751	9748	1	1	2	3	3
13	9744	9740	9736	9732	9728	9724	9720	9715	9711	9707	1	1	2	3	3
14	9703	9699	9694	9689	9686	9681	9677	9673	9668	9664	1	1	2	3	4
15	9659	9655	9650	9646	9641	9636	9632	9627	9622	9617	1	2	2	3	4
16	9613	9608	9603	9598	9593	9588	9583	9578	9573	9568	1	2	2	3	4
17	9563	9558	9553	9548	9542	9537	9532	9527	9521	9516	1	2	3	4	4
18	9511	9506	9500	9494	9489	9483	9478	9472	9466	9461	1	2	3	4	5
19	9455	9449	9444	9438	9432	9426	9421	9415	9409	9403	1	2	3	4	5
20	9397	9391	9385	9379	9373	9367	9361	9354	9348	9342	1	2	3	4	5
21	9336	9330	9323	9317	9311	9304	9298	9291	9285	9278	1	2	3	4	5
22	9272	9265	9259	9252	9245	9239	9232	9225	9219	9212	1	2	3	4	6
23	9205	9198	9191	9184	9178	9171	9164	9157	9150	9143	1	2	3	5	6
24	9135	9128	9121	9114	9107	9100	9092	9085	9078	9070	1	2	4	5	6
25	9063	9056	9048	9041	9033	9026	9018	9011	9003	8996	1	3	4	5	6
26	8988	8980	8973	8965	8957	8949	8942	8934	8926	8918	1	3	4	5	6
27	8910	8902	8894	8886	8878	8870	8862	8854	8846	8838	1	3	4	5	7
28	8829	8821	8813	8805	8796	8788	8780	8771	8763	8755	1	3	4	6	7
29	8746	8738	8729	8721	8712	8704	8695	8686	8678	8669	1	3	4	6	7
30	8660	8652	8643	8634	8625	8616	8607	8599	8590	8581	1	3	4	6	7
31	8572	8563	8554	8545	8536	8526	8517	8508	8499	8490	2	3	5	6	8
32	8480	8471	8462	8453	8443	8434	8425	8415	8406	8396	2	3	5	6	8
33	8387	8377	8368	8358	8348	8339	8329	8320	8310	8300	2	3	5	6	8
34	8290	8281	8271	8261	8251	8241	8231	8221	8211	8202	2	3	5	7	8
35	8192	8181	8171	8161	8151	8141	8131	8121	8111	8100	2	3	5	7	8
36	8090	8080	8070	8060	8049	8039	8028	8018	8007	7997	2	3	5	7	9
37	7986	7976	7965	7955	7944	7934	7923	7912	7902	7891	2	4	5	7	9
38	7880	7869	7859	7848	7837	7826	7815	7804	7793	7782	2	4	5	7	9
39	7771	7760	7749	7738	7727	7716	7705	7694	7683	7672	2	4	6	7	9
40	7660	7649	7638	7627	7615	7604	7593	7581	7570	7559	2	4	6	8	9
41	7547	7536	7524	7513	7501	7490	7478	7466	7455	7443	2	4	6	8	10
42	7431	7420	7408	7396	7385	7373	7361	7349	7337	7325	2	4	6	8	10
43	7314	7302	7290	7278	7266	7254	7242	7230	7218	7206	2	4	6	8	10
44	7198	7181	7169	7157	7145	7133	7120	7108	7096	7083	2	4	6	8	10

# REDPATH, BROWN & CO., LIMITED.

## NATURAL COSINES.

N.B.—Subtract Mean Differences.

	0'	6'	12'	18'	24'	30'	36'	42'	48'	54'	Mean Differences.				
											1'	2'	3'	4'	5'
45	7071	7059	7046	7034	7022	7009	6997	6984	6972	6959	2	4	6	8	10
46	6947	6934	6921	6909	6896	6884	6871	6858	6846	6833	2	4	6	8	11
47	6820	6807	6794	6782	6769	6756	6743	6730	6717	6704	2	4	6	9	11
48	6691	6678	6665	6652	6639	6626	6613	6600	6587	6574	2	4	7	9	11
49	6561	6547	6534	6521	6508	6494	6481	6468	6455	6441	2	4	7	9	11
50	6428	6414	6401	6388	6374	6361	6347	6334	6320	6307	2	4	7	9	11
51	6293	6280	6266	6252	6239	6225	6211	6198	6184	6170	2	5	7	9	11
52	6157	6143	6129	6115	6101	6088	6074	6060	6046	6032	2	5	7	9	12
53	6018	6004	5990	5976	5962	5948	5934	5920	5906	5892	2	5	7	9	12
54	5878	5864	5850	5835	5821	5807	5793	5779	5764	5750	2	5	7	9	12
55	5736	5721	5707	5693	5678	5664	5650	5635	5621	5606	2	5	7	10	12
56	5592	5577	5563	5548	5534	5519	5505	5490	5476	5461	2	5	7	10	12
57	5446	5432	5417	5402	5388	5373	5358	5344	5329	5314	2	5	7	10	12
58	5299	5284	5270	5255	5240	5225	5210	5195	5180	5165	2	5	7	10	12
59	5150	5135	5120	5105	5090	5075	5060	5045	5030	5015	3	5	8	10	13
60	5000	4985	4970	4955	4939	4924	4909	4894	4879	4863	3	5	8	10	13
61	4848	4833	4818	4802	4787	4772	4756	4741	4726	4710	3	5	8	10	13
62	4695	4679	4664	4648	4633	4617	4602	4586	4571	4555	3	5	8	10	13
63	4540	4524	4509	4493	4478	4462	4446	4431	4415	4399	3	5	8	10	13
64	4384	4368	4352	4337	4321	4305	4289	4274	4258	4242	3	5	8	11	13
65	4228	4210	4195	4179	4163	4147	4131	4115	4099	4083	3	5	8	11	13
66	4067	4051	4035	4019	4003	3987	3971	3955	3939	3923	3	5	8	11	14
67	3907	3891	3875	3859	3843	3827	3811	3795	3778	3762	3	5	8	11	14
68	3746	3730	3714	3697	3681	3665	3649	3633	3616	3600	3	5	8	11	14
69	3584	3567	3551	3535	3518	3502	3486	3469	3453	3437	3	5	8	11	14
70	3420	3404	3387	3371	3355	3338	3322	3305	3289	3272	3	5	8	11	14
71	3256	3239	3223	3206	3190	3173	3156	3140	3123	3107	3	5	8	11	14
72	3090	3074	3057	3040	3024	3007	2990	2974	2957	2940	3	5	8	11	14
73	2924	2907	2890	2874	2857	2840	2823	2807	2790	2773	3	5	8	11	14
74	2756	2740	2723	2706	2689	2672	2655	2639	2622	2606	3	5	8	11	14
75	2588	2571	2554	2538	2521	2504	2487	2470	2453	2436	3	5	8	11	14
76	2419	2402	2385	2368	2351	2334	2317	2300	2284	2267	3	5	8	11	14
77	2250	2233	2216	2198	2181	2164	2147	2130	2113	2096	3	5	9	11	14
78	2079	2062	2045	2028	2011	1994	1977	1959	1942	1925	3	5	9	11	14
79	1908	1891	1874	1857	1840	1822	1805	1788	1771	1754	3	5	9	12	14
80	1736	1719	1702	1685	1668	1650	1633	1616	1599	1582	3	5	9	12	14
81	1564	1547	1530	1513	1495	1478	1461	1444	1426	1409	3	5	9	12	14
82	1392	1374	1357	1340	1323	1305	1288	1271	1253	1236	3	5	9	12	14
83	1219	1201	1184	1167	1149	1132	1115	1097	1080	1063	3	5	9	12	14
84	1046	1028	1011	993	976	958	941	924	906	889	3	5	9	12	14
85	872	854	837	819	802	785	767	750	732	715	3	5	9	12	14
86	698	680	663	645	628	610	593	576	558	541	3	5	9	12	15
87	523	506	488	471	454	436	419	401	384	366	3	5	9	12	15
88	349	332	314	297	279	262	244	227	209	192	3	5	9	12	15
89	175	157	140	122	105	87	70	52	35	17	3	5	9	12	15

# REDPATH, BROWN & CO., LIMITED.

## NATURAL TANGENTS.

	0'	6'	12'	18'	24'	30'	36'	42'	48'	54'	Mean Differences.				
											1'	2'	3'	4'	5'
0	0000	0017	0035	0052	0070	0087	0105	0122	0140	0157	3	6	9	12	15
1	0175	0192	0209	0227	0244	0262	0279	0297	0314	0332	3	6	9	12	15
2	0349	0367	0384	0402	0419	0437	0454	0472	0489	0507	3	6	9	12	15
3	0524	0542	0559	0577	0594	0612	0629	0647	0664	0682	3	6	9	12	15
4	0699	0717	0734	0752	0769	0787	0806	0822	0840	0857	3	6	9	12	15
5	0875	0892	0910	0928	0946	0963	0981	0998	1016	1033	3	6	9	12	15
6	1051	1069	1086	1104	1122	1139	1157	1175	1192	1210	3	6	9	12	15
7	1228	1246	1263	1281	1299	1317	1334	1352	1370	1388	3	6	9	12	15
8	1405	1423	1441	1459	1477	1495	1512	1530	1548	1566	3	6	9	12	15
9	1584	1602	1620	1638	1655	1673	1691	1709	1727	1745	3	6	9	12	15
10	1763	1781	1799	1817	1835	1853	1871	1889	1908	1926	3	6	9	12	15
11	1944	1962	1980	1998	2016	2035	2053	2071	2089	2107	3	6	9	12	15
12	2126	2144	2162	2180	2199	2217	2235	2254	2272	2290	3	6	9	12	15
13	2309	2327	2345	2364	2382	2401	2419	2438	2456	2475	3	6	9	12	15
14	2493	2512	2530	2549	2568	2586	2605	2623	2642	2661	3	6	9	12	15
15	2679	2698	2717	2736	2754	2773	2792	2811	2830	2849	3	6	9	13	16
16	2867	2886	2905	2924	2943	2962	2981	3000	3019	3038	3	6	9	13	16
17	3057	3076	3095	3115	3134	3153	3172	3191	3211	3230	3	6	10	13	16
18	3249	3269	3288	3307	3327	3346	3365	3385	3404	3424	3	6	10	13	16
19	3443	3463	3482	3502	3522	3541	3561	3581	3600	3620	3	7	10	13	16
20	3640	3659	3679	3699	3719	3739	3759	3779	3799	3819	3	7	10	13	17
21	3839	3859	3879	3899	3919	3939	3959	3979	4000	4020	3	7	10	13	17
22	4040	4061	4081	4101	4122	4142	4163	4183	4204	4224	3	7	10	14	17
23	4245	4265	4286	4307	4327	4348	4369	4390	4411	4431	3	7	10	14	17
24	4452	4473	4494	4515	4536	4557	4578	4599	4621	4642	4	7	11	14	18
25	4663	4684	4706	4727	4748	4770	4791	4813	4834	4856	4	7	11	14	18
26	4877	4899	4921	4942	4964	4986	5008	5029	5051	5073	4	7	11	15	18
27	5095	5117	5139	5161	5184	5206	5228	5250	5272	5295	4	7	11	15	18
28	5317	5340	5362	5384	5407	5430	5452	5475	5498	5520	4	8	11	15	19
29	5543	5566	5589	5612	5635	5658	5681	5704	5727	5750	4	8	12	15	19
30	5774	5797	5820	5844	5867	5890	5914	5938	5961	5985	4	8	12	16	20
31	6009	6032	6056	6080	6104	6128	6152	6176	6200	6224	4	8	12	16	20
32	6249	6273	6297	6322	6346	6371	6395	6420	6445	6469	4	8	12	16	20
33	6494	6519	6544	6569	6594	6619	6644	6669	6694	6720	4	8	13	17	21
34	6745	6771	6796	6822	6847	6873	6899	6924	6950	6976	4	9	13	17	21
35	7002	7028	7054	7080	7107	7133	7159	7186	7212	7239	4	9	13	18	22
36	7265	7292	7319	7346	7373	7400	7427	7454	7481	7508	5	9	14	18	23
37	7536	7563	7590	7618	7646	7673	7701	7729	7757	7785	5	9	14	18	23
38	7813	7841	7869	7898	7926	7954	7983	8012	8040	8069	5	9	14	19	24
39	8098	8127	8156	8185	8214	8243	8273	8302	8332	8361	5	10	15	20	24
40	8391	8421	8451	8481	8511	8541	8571	8601	8632	8662	5	10	15	20	25
41	8696	8724	8754	8785	8816	8847	8878	8910	8941	8972	5	10	16	21	26
42	9004	9036	9067	9099	9131	9163	9195	9228	9260	9293	5	11	16	21	27
43	9325	9358	9391	9424	9457	9490	9523	9556	9590	9623	6	11	17	22	28
44	9657	9691	9725	9759	9793	9827	9861	9896	9930	9965	6	11	17	23	29

# REDPATH, BROWN & CO., LIMITED.

## NATURAL TANGENTS.

	0'	6'	12'	18'	24'	30'	36'	42'	48'	54'	Mean Differences.				
											1'	2'	3'	4'	5'
45	1-0000	0035	0070	0105	0141	0176	0212	0247	0283	0319	6	12	18	24	30
46	1-0355	0392	0428	0464	0501	0538	0575	0612	0649	0686	6	12	18	25	31
47	1-0724	0761	0799	0837	0875	0913	0951	0990	1028	1067	6	13	19	25	32
48	1-1106	1145	1184	1224	1263	1303	1343	1383	1423	1463	7	13	20	27	33
49	1-1504	1544	1585	1626	1667	1708	1749	1792	1833	1875	7	14	21	28	34
50	1-1918	1960	2002	2045	2088	2131	2174	2218	2261	2305	7	14	22	29	36
51	1-2349	2393	2437	2482	2527	2572	2617	2662	2708	2753	8	15	23	30	38
52	1-2799	2846	2892	2938	2985	3032	3079	3127	3175	3222	8	16	24	31	39
53	1-3270	3319	3367	3416	3465	3514	3563	3613	3663	3713	8	16	25	33	41
54	1-3764	3814	3865	3916	3968	4019	4071	4124	4176	4229	9	17	26	34	43
55	1-4281	4335	4388	4442	4496	4550	4605	4659	4715	4770	9	18	27	36	45
56	1-4826	4882	4938	4994	5051	5108	5166	5224	5282	5340	10	19	29	38	48
57	1-5399	5458	5517	5577	5637	5697	5757	5818	5880	5941	10	20	30	40	50
58	1-6003	6066	6128	6191	6255	6319	6383	6447	6512	6577	11	21	32	43	53
59	1-6643	6709	6775	6842	6909	6977	7045	7113	7182	7251	11	23	34	45	56
60	1-7321	7391	7461	7532	7603	7675	7747	7820	7893	7966	12	24	36	48	60
61	1-8040	8115	8190	8265	8341	8418	8495	8572	8650	8728	13	25	38	51	64
62	1-8807	8887	8967	9047	9128	9210	9292	9375	9458	9542	14	27	41	55	69
63	1-9626	9711	9797	9883	9970	10057	10145	10233	10323	10413	15	29	44	59	73
64	2-0503	0594	0686	0778	0872	0965	1060	1155	1251	1348	16	31	47	63	78
65	2-1445	1543	1642	1742	1842	1943	2045	2148	2251	2355	17	34	51	68	85
66	2-2480	2566	2673	2781	2889	2998	3109	3220	3332	3445	18	37	55	73	92
67	2-3569	3673	3789	3906	4023	4142	4262	4383	4504	4627	20	40	60	79	99
68	2-4751	4870	5002	5129	5257	5386	5517	5649	5782	5916	22	43	66	87	108
69	2-6061	6187	6325	6464	6605	6746	6889	7034	7179	7326	24	47	71	95	119
70	2-7475	7625	7776	7929	8083	8239	8397	8556	8716	8878	26	52	78	104	131
71	2-9042	9208	9376	9544	9714	9887	10061	10237	10415	10595	29	58	87	116	145
72	3-0777	0961	1146	1334	1524	1716	1910	2106	2306	2506	32	64	96	129	161
73	3-2709	2914	3122	3332	3544	3769	3977	4197	4420	4646	36	72	108	144	180
74	3-4874	5106	5339	5576	5816	6059	6305	6554	6806	7062	41	81	122	163	204
75	3-7821	7583	7848	8118	8391	8667	8947	9232	9520	9812	Owing to the rapidity with which the tangent changes, mean differences cease to be useful.				
76	4-0108	0408	0713	1022	1335	1653	1976	2303	2635	2972					
77	4-2316	2662	3016	3374	3737	4107	4483	4864	5250	5640					
78	4-7046	7453	7867	8288	8716	9152	9594	10045	10504	10970					
79	5-1446	1929	2422	2924	3435	3956	4486	5026	5578	6140					
80	5-6713	7297	7894	8502	9124	9758	10405	1066	1742	2422					
81	6-3138	3859	4596	5350	6122	6912	7720	8548	9395	10264					
82	7-1164	8066	8902	9762	10647	11558	12495	13458	14447	15462					
83	8-1443	2686	3363	4062	4782	5523	6285	7068	7872	8697					
84	9-514	9-677	9-845	10-02	10-20	10-39	10-58	10-79	10-99	11-20					
85	11-43	11-66	11-91	12-16	12-43	12-71	13-00	13-30	13-62	13-96					
86	14-20	14-67	15-06	15-46	15-89	16-35	16-83	17-34	17-89	18-46					
87	19-08	19-74	20-45	21-20	22-02	22-90	23-86	24-90	26-03	27-27					
88	28-64	30-14	31-82	33-69	35-80	38-19	40-92	44-07	47-74	52-06					
89	57-29	63-06	71-62	81-85	95-49	114-6	143-2	191-0	286-5	578-0					

# REDPATH, BROWN & CO., LIMITED.

## NATURAL COSECANTS.

N.B.—Subtract Mean Differences.

	0'	6'	12'	18'	24'	30'	36'	42'	48'	54'	Mean Differences.				
											1'	2'	3'	4'	5'
0'	Inf.	578.0	286.5	191.0	143.2	114.6	95.49	81.85	71.82	63.06	Owing to the rapidity with which cosecant changes, mean differences are of no use.				
1	57.30	52.09	47.75	44.08	40.93	38.20	35.81	33.71	31.84	30.16					
2	38.65	27.29	26.05	24.92	23.88	22.93	22.04	21.23	20.47	19.77					
3	19.11	18.49	17.91	17.37	16.86	16.38	15.93	15.50	15.09	14.70					
4	14.34	13.99	13.65	13.34	13.03	12.75	12.47	12.20	11.95	11.71	Owing to the rapidity with which cosecant changes, mean differences are of no use.				
5	11.47	11.25	11.03	10.83	10.63	10.43	10.25	10.07	9.895	9.728					
6	9.5668	4.105	2593	1129	9711	8337	7004	6711	4467	3233					
7	8.2056	0905	9787	8700	7642	6613	5611	4635	3634	2757					
8	7.1353	0972	0112	9273	8454	7655	6874	6111	5366	4637	Owing to the rapidity with which cosecant changes, mean differences are of no use.				
9	6.8926	8228	2546	1880	1227	0589	9963	9351	8751	8164					
10	5.7588	7023	6470	5928	5396	4874	4362	3860	3367	2883					
11	5.2408	1942	1484	1034	0503	0159	9732	9313	8901	8495	61	122	182	243	304
12	4.9097	7706	7321	6942	6569	6202	5841	5486	5137	4793	52	104	156	208	260
13	4.4454	4121	3792	3469	3150	2837	2527	2223	1923	1627	45	90	135	180	225
14	4.1336	1048	0786	0486	0211	9939	9672	9408	9147	8890	Owing to the rapidity with which cosecant changes, mean differences are of no use.				
15	3.8637	8387	8140	7897	7657	7420	7186	6955	6727	6502					
16	3.6280	6060	5843	5629	5418	5209	5003	4790	4598	4399	39	79	118	157	196
17	3.4203	4009	3817	3628	3440	3255	3072	2891	2712	2536	35	60	104	133	173
18	3.2361	2188	2017	1848	1681	1515	1352	1190	1030	0872	31	61	92	123	154
19	3.0716	0561	0407	0256	0106	9957	9811	9665	9521	9379	27	55	82	110	137
20	2.9238	9099	8900	8824	8688	8555	8422	8291	8161	8032	25	49	74	99	123
21	2.7904	7778	7653	7529	7407	7285	7165	7046	6927	6811	22	44	67	89	111
22	2.6695	6580	6406	6242	6131	6022	5913	5805	5699	5595	20	40	60	81	101
23	2.5593	5488	5384	5282	5180	5078	4978	4879	4780	4683	18	37	55	73	92
24	2.4586	4490	4395	4300	4207	4114	4022	3931	3841	3751	17	34	50	67	84
25	2.3662	3574	3486	3400	3314	3228	3144	3060	2976	2894	15	31	46	62	77
26	2.2812	2730	2650	2570	2490	2412	2333	2256	2179	2103	14	28	43	57	71
27	2.2027	1952	1877	1803	1730	1657	1584	1513	1441	1371	13	26	39	52	65
28	2.1301	1231	1162	1093	1025	0957	0890	0824	0757	0692	12	24	36	48	60
29	2.0627	0562	0498	0434	0371	0308	0245	0188	0132	0061	10	21	31	42	52
30	2.0000	99.0	9880	9851	9812	9763	9703	9645	9587	9530	10	19	29	39	49
31	1.9416	9360	9304	9249	9194	9139	9084	9031	8977	8924	9	18	27	36	45
32	1.8871	8818	8766	8714	8663	8612	8561	8510	8460	8410	8	17	25	34	42
33	1.8361	8312	8263	8214	8166	8118	8070	8023	7976	7929	8	16	24	32	40
34	1.7883	7837	7791	7745	7700	7655	7610	7565	7522	7478	7	15	22	30	37
35	1.7434	7391	7348	7305	7263	7221	7179	7137	7095	7054	7	14	21	28	35
36	1.7013	6972	6932	6892	6852	6812	6772	6733	6694	6655	7	13	20	26	33
37	1.6616	6578	6540	6502	6464	6427	6390	6353	6316	6279	6	12	19	25	31
38	1.6243	6247	6171	6135	6099	6064	6029	5994	5959	5925	6	12	18	23	29
39	1.5890	5856	5822	5788	5755	5721	5688	5655	5622	5590	6	11	17	22	28
40	1.5557	5525	5493	5461	5429	5398	5366	5335	5304	5273	5	10	16	21	26
41	1.5243	5212	5182	5151	5121	5092	5062	5032	5003	4974	5	10	15	20	25
42	1.4945	4916	4887	4859	4830	4802	4774	4746	4718	4690	5	9	14	19	23
43	1.4663	4635	4608	4581	4554	4527	4501	4474	4448	4422	4	9	13	18	22
44	1.4396	4370	4344	4318	4293	4267	4242	4217	4192	4167	4	8	12	17	21

# REDPATH, BROWN & CO., LIMITED.

## NATURAL COSECANTS.

N.B.—Subtract Mean Differences.

	0'	6'	12'	18'	24'	30'	36'	42'	48'	54'	Mean Differences.				
											1'	2'	3'	4'	5'
45	1'4142	4118	4093	4069	4044	4020	3996	3972	3949	3925	4	8	12	16	20
46	1'3902	3878	3855	3832	3809	3786	3763	3741	3718	3696	4	8	11	15	19
47	1'3673	3651	3629	3607	3585	3563	3542	3520	3499	3478	4	7	11	14	18
48	1'3456	3435	3414	3393	3373	3352	3331	3311	3291	3270	3	7	10	14	17
49	1'3250	3230	3210	3190	3171	3151	3131	3112	3093	3073	3	7	10	13	16
50	1'3054	3035	3016	2997	2978	2960	2941	2923	2904	2886	3	6	9	12	15
51	1'2863	2845	2827	2810	2792	2775	2758	2742	2725	2708	3	6	9	12	15
52	1'2690	2673	2656	2639	2622	2605	2588	2571	2554	2538	3	6	8	11	14
53	1'2521	2505	2489	2472	2456	2440	2424	2408	2392	2376	3	5	8	11	13
54	1'2361	2345	2329	2314	2299	2283	2268	2253	2238	2223	3	5	8	10	13
55	1'2208	2193	2178	2163	2149	2134	2120	2105	2091	2078	2	5	7	10	12
56	1'2062	2048	2034	2020	2006	1992	1978	1964	1951	1937	2	5	7	9	12
57	1'1924	1910	1897	1883	1870	1857	1844	1831	1818	1805	2	4	7	9	11
58	1'1792	1779	1766	1753	1741	1728	1716	1703	1691	1679	2	4	6	8	10
59	1'1666	1654	1642	1630	1618	1606	1594	1582	1570	1559	2	4	6	8	10
60	1'1547	1535	1524	1512	1501	1490	1478	1467	1456	1445	2	4	6	8	9
61	1'1434	1423	1412	1401	1390	1379	1368	1357	1347	1336	2	4	5	7	9
62	1'1326	1315	1305	1294	1284	1274	1263	1253	1243	1233	2	3	5	7	9
63	1'1223	1213	1203	1194	1184	1174	1165	1155	1145	1136	2	3	5	6	8
64	1'1126	1117	1107	1098	1089	1079	1070	1061	1052	1043	2	3	5	6	8
65	1'1034	1025	1016	1007	998	989	981	972	963	955	1	3	4	6	7
66	1'0946	0938	0929	0921	0913	0904	0896	0888	0880	0872	1	3	4	6	7
67	1'0864	0856	0848	0840	0832	0824	0816	0808	0801	0793	1	3	4	5	7
68	1'0785	0778	0770	0763	0755	0748	0740	0733	0726	0719	1	2	4	5	6
69	1'0711	0704	0697	0690	0683	0676	0669	0662	0655	0649	1	2	3	5	6
70	1'0642	0635	0628	0622	0615	0608	0602	0595	0589	0583	1	2	3	4	5
71	1'0579	0570	0564	0557	0551	0545	0539	0533	0527	0521	1	2	3	4	5
72	1'0516	0509	0503	0497	0491	0485	0479	0474	0468	0463	1	2	3	4	5
73	1'0457	0451	0446	0440	0435	0429	0424	0419	0413	0408	1	2	3	4	4
74	1'0403	0398	0393	0388	0382	0377	0372	0367	0362	0358	1	2	2	3	4
75	1'0353	0348	0343	0338	0334	0329	0324	0320	0315	0311	1	2	2	3	4
76	1'0309	0302	0297	0293	0288	0284	0280	0276	0271	0267	1	1	2	3	4
77	1'0263	0259	0255	0251	0247	0243	0239	0235	0231	0227	1	1	2	3	3
78	1'0223	0220	0216	0212	0209	0205	0201	0198	0194	0191	1	1	2	3	3
79	1'0187	0184	0180	0177	0174	0170	0167	0164	0161	0157	1	1	2	2	3
80	1'0154	0151	0148	0145	0142	0139	0136	0133	0130	0127	0	1	1	2	2
81	1'0125	0122	0119	0116	0114	0111	0108	0106	0103	0101	0	1	1	2	2
82	1'0093	0096	0093	0091	0089	0086	0084	0082	0079	0077	0	1	1	2	2
83	1'0075	0073	0071	0069	0067	0065	0063	0061	0059	0057	0	1	1	1	2
84	1'0065	0063	0061	0060	0058	0056	0054	0051	0049	0047	0	1	1	1	1
85	1'0038	0037	0035	0034	0032	0031	0030	0028	0027	0026	0	0	1	1	1
86	1'0024	0023	0022	0021	0020	0019	0018	0017	0016	0015	0	0	0	1	1
87	1'0014	0013	0012	0011	0010	0010	0009	0008	0007	0007	0	0	0	1	1
88	1'0006	0006	0005	0004	0004	0003	0003	0003	0002	0002	0	0	0	0	0
89	1'0002	0001	0001	0001	0001	0000	0000	0000	0000	0000	0	0	0	0	0

# REDPATH, BROWN & CO., LIMITED.

## NATURAL SECANTS.

	0'	6'	12'	18'	24'	30'	36'	42'	48'	54'	Mean Differences.				
											1'	2'	3'	4'	5'
0°	1.0000	0000	0000	0000	0000	0000	0001	0001	0001	0001	0	0	0	0	0
1	1.0002	0002	0002	0003	0003	0003	0004	0004	0005	0006	0	0	0	0	0
2	1.0006	0007	0007	0008	0009	0010	0010	0011	0012	0013	0	0	0	0	0
3	1.0014	0015	0016	0017	0018	0019	0020	0021	0022	0023	0	0	1	1	1
4	1.0024	0026	0027	0028	0030	0031	0032	0034	0035	0037	0	0	1	1	1
5	1.0038	0040	0041	0043	0045	0046	0048	0050	0051	0053	0	1	1	1	1
6	1.0055	0057	0059	0061	0063	0065	0067	0069	0071	0073	0	1	1	1	2
7	1.0075	0077	0079	0082	0084	0086	0089	0091	0093	0096	0	1	1	2	2
8	1.0098	0101	0103	0106	0108	0111	0114	0116	0119	0122	0	1	1	2	2
9	1.0125	0127	0130	0133	0136	0139	0142	0145	0148	0151	0	1	1	2	2
10	1.0154	0157	0161	0164	0167	0170	0174	0177	0180	0184	1	1	2	2	3
11	1.0187	0191	0194	0198	0201	0205	0209	0212	0216	0220	1	1	2	3	3
12	1.0223	0227	0231	0235	0239	0243	0247	0251	0255	0259	1	1	2	3	3
13	1.0263	0267	0271	0276	0280	0284	0288	0293	0297	0302	1	1	2	3	4
14	1.0306	0311	0315	0320	0324	0329	0334	0338	0343	0348	1	2	2	3	4
15	1.0353	0358	0363	0367	0372	0377	0382	0388	0393	0398	1	2	3	3	4
16	1.0403	0408	0413	0419	0424	0429	0435	0440	0446	0451	1	2	3	4	5
17	1.0457	0463	0468	0474	0480	0485	0491	0497	0503	0509	1	2	3	4	5
18	1.0515	0521	0527	0533	0539	0545	0551	0557	0564	0570	1	2	3	4	5
19	1.0576	0583	0589	0595	0602	0608	0615	0622	0628	0636	1	2	3	4	5
20	1.0642	0649	0655	0662	0669	0676	0683	0690	0697	0704	1	2	3	5	6
21	1.0711	0719	0726	0733	0740	0748	0755	0763	0770	0778	1	2	4	5	6
22	1.0785	0793	0801	0808	0816	0824	0832	0840	0848	0856	1	3	4	5	6
23	1.0864	0872	0880	0888	0896	0904	0913	0921	0929	0938	1	3	4	6	7
24	1.0946	0955	0963	0972	0981	0989	0998	1007	1016	1025	1	3	4	6	7
25	1.1034	1043	1052	1061	1070	1079	1089	1098	1107	1117	2	3	5	6	8
26	1.1126	1136	1145	1155	1164	1174	1184	1194	1203	1213	2	3	5	6	8
27	1.1223	1233	1243	1253	1264	1274	1284	1294	1305	1315	2	3	5	7	9
28	1.1326	1336	1347	1357	1368	1379	1390	1401	1412	1423	2	4	5	7	9
29	1.1434	1445	1456	1467	1478	1490	1501	1512	1524	1535	2	4	6	8	9
30	1.1547	1559	1570	1582	1594	1606	1618	1630	1642	1654	2	4	6	8	10
31	1.1666	1679	1691	1703	1716	1728	1741	1753	1766	1779	2	4	6	8	10
32	1.1792	1805	1818	1831	1844	1857	1870	1883	1897	1910	2	4	7	9	11
33	1.1924	1937	1951	1964	1978	1992	2006	2020	2034	2048	2	5	7	9	12
34	1.2062	2076	2091	2105	2120	2134	2149	2163	2178	2193	2	5	7	10	12
35	1.2208	2223	2238	2253	2268	2283	2299	2314	2329	2345	3	5	8	10	13
36	1.2361	2376	2392	2408	2424	2440	2456	2472	2489	2505	3	5	8	11	13
37	1.2521	2538	2554	2571	2588	2605	2622	2639	2656	2673	3	6	8	11	14
38	1.2690	2708	2725	2742	2760	2778	2796	2813	2831	2849	3	6	9	12	15
39	1.2868	2886	2904	2923	2941	2960	2978	2997	3016	3035	3	6	9	12	16
40	1.3054	3073	3093	3112	3131	3151	3171	3190	3210	3230	3	7	10	13	16
41	1.3250	3270	3291	3311	3331	3352	3373	3393	3414	3435	3	7	10	14	17
42	1.3456	3478	3499	3520	3542	3563	3585	3607	3629	3651	4	7	11	14	18
43	1.3673	3696	3718	3741	3763	3786	3809	3832	3855	3878	4	8	11	15	19
44	1.3902	3925	3949	3972	3996	4020	4044	4069	4093	4118	4	8	12	16	20

# REDPATH, BROWN & CO., LIMITED.

## NATURAL SECANTS.

	0'	6'	12'	18'	24'	30'	36'	42'	48'	54'	Mean Differences.				
											1'	2'	3'	4'	5'
45°	1'4142	4167	4192	4217	4242	4267	4293	4318	4344	4370	4	8	13	17	21
46	1'4396	4422	4448	4474	4501	4527	4554	4581	4608	4635	4	9	13	18	22
47	1'4603	4690	4718	4746	4774	4802	4830	4859	4887	4916	5	9	14	19	23
48	1'4945	4974	5003	5032	5062	5092	5121	5151	5182	5212	5	10	15	20	25
49	1'5243	5273	5304	5335	5366	5398	5429	5461	5493	5525	5	10	16	21	26
50	1'5557	5590	5622	5655	5688	5721	5755	5788	5822	5856	6	11	17	22	28
51	1'5890	5925	5969	5994	6029	6064	6099	6135	6171	6207	6	12	18	24	29
52	1'6243	6279	6316	6353	6390	6427	6464	6502	6540	6578	6	12	19	25	31
53	1'6616	6655	6694	6733	6772	6812	6852	6892	6932	6972	7	13	20	26	33
54	1'7013	7054	7095	7137	7179	7221	7263	7305	7348	7391	7	14	21	28	35
55	1'7434	7478	7522	7566	7610	7655	7700	7745	7791	7837	7	15	22	30	37
56	1'7883	7929	7976	8023	8070	8118	8166	8214	8263	8312	8	16	24	32	40
57	1'8361	8410	8460	8510	8561	8612	8663	8714	8766	8818	9	17	26	34	43
58	1'8871	8924	8977	9031	9084	9139	9194	9249	9304	9360	9	18	27	36	45
59	1'9416	9473	9530	9587	9646	9703	9762	9821	9880	9940	10	19	29	39	49
60	2'0000	0061	0122	0183	0245	0308	0371	0434	0498	0562	10	21	31	42	52
61	2'0627	0692	0757	0824	0890	0957	1025	1093	1162	1231	11	22	34	45	56
62	2'1301	1371	1441	1513	1584	1657	1730	1803	1877	1952	12	24	36	48	61
63	2'2027	2103	2179	2256	2333	2412	2490	2570	2650	2730	13	26	39	52	66
64	2'2812	2894	2976	3060	3144	3228	3314	3400	3486	3574	14	28	43	57	71
65	2'3662	3751	3841	3931	4022	4114	4207	4300	4395	4490	15	31	46	62	77
66	2'4586	4683	4780	4879	4978	5078	5180	5282	5384	5488	17	34	50	67	84
67	2'5593	5699	5806	5913	6022	6131	6242	6354	6466	6580	18	37	55	73	92
68	2'6695	6811	6927	7046	7165	7285	7407	7529	7653	7778	20	40	60	81	101
69	2'7904	8032	8161	8291	8422	8556	8688	8824	8960	9099	22	44	67	89	111
70	2'9238	9379	9521	9665	9811	9957	0106	0256	0407	0561	25	49	74	99	123
71	3'0716	0872	1030	1190	1352	1515	1681	1848	2017	2188	27	55	82	110	137
72	3'2361	2536	2712	2891	3072	3255	3440	3628	3817	4009	31	61	92	123	154
73	3'4203	4399	4606	4799	5003	5209	5418	5629	5843	6060	35	69	104	138	173
74	3'6280	6592	6727	6955	7186	7420	7657	7897	8140	8387	39	79	118	167	196
75	3'8637	8890	9147	9408	9672	9939	0211	0486	0765	1048	45	90	135	180	225
76	4'1386	1027	1223	1423	1627	1837	2050	2268	2488	2712	52	104	166	208	260
77	4'4454	4793	5137	5486	5841	6202	6569	6942	7321	7706	61	122	182	243	304
78	4'8097	8496	8901	9313	9732	10159	10593	11034	11484	11943	72	144	216	287	359
79	5'2408	2883	3367	3860	4362	4874	5396	5928	6470	7023	86	173	259	345	432
80	5'7688	8164	8651	9151	9663	10189	10727	11268	11823	12383	100	200	300	400	500
81	6'3925	4637	5363	6111	6874	7655	8454	9273	10112	10972	120	240	360	480	600
82	7'1853	2757	3684	4636	5611	6613	7642	8700	9787	10905	140	280	420	560	700
83	8'2055	8338	4457	5711	7004	8337	9711	11129	12593	14105	160	320	480	640	800
84	9'5668	7283	8965	10686	12477	14334	16261	18260	20333	22483	180	360	540	720	900
85	11'47	11'71	11'95	12'20	12'47	12'75	13'03	13'34	13'65	13'99	Owing to the rapidity with which the secant changes, mean differences cease to be useful.				
86	14'34	14'70	15'09	15'50	15'93	16'38	16'86	17'37	17'91	18'49					
87	19'11	19'77	20'47	21'23	22'04	22'93	23'88	24'92	26'05	27'29					
88	28'55	30'16	31'34	33'01	34'38	36'20	38'08	40'08	42'25	44'59					
89	57'30	59'06	61'02	63'15	65'49	68'14	71'00	74'00	77'25	80'80					

# REDPATH, BROWN & CO., LIMITED.

## NATURAL COTANGENTS.

N.B.—Subtract Mean Differences.

	0'	6'	12'	18'	24'	30'	36'	42'	48'	54'	Mean Differences.				
											1'	2'	3'	4'	5'
0'	Inf.	573.0	286.5	191.0	143.2	114.6	95.49	81.85	71.92	63.66	Owing to the rapidity with which cotangent changes, mean differences are of no use.				
1	57.29	62.08	47.74	44.07	40.92	38.19	35.80	33.69	31.82	30.14					
2	28.64	27.27	26.03	24.90	23.86	22.90	22.02	21.20	20.45	19.74					
3	19.08	18.46	17.89	17.34	16.83	16.35	15.89	15.46	15.06	14.67					
4	14.30	13.95	13.62	13.30	13.00	12.71	12.43	12.16	11.91	11.66					
5	11.43	11.20	10.99	10.78	10.58	10.39	10.20	10.02	9.846	9.677	Owing to the rapidity with which cotangent changes, mean differences are of no use.				
6	9.5144	3572	2052	0579	9152	7769	6427	5126	3863	2636					
7	8.1443	0285	9158	8062	6966	5858	4947	3962	3002	2066					
8	7.1154	0264	9395	8548	7720	6912	6122	5350	4596	3859					
9	6.3138	2432	1742	1066	0405	9758	9124	8502	7894	7297					
10	5.6713	0140	5578	5026	4486	3955	3435	2924	2422	1929	Owing to the rapidity with which cotangent changes, mean differences are of no use.				
11	5.1446	0970	0504	0045	9594	9152	8716	8288	7867	7453					
12	4.7046	6646	6252	5864	5483	5107	4737	4374	4015	3662					
13	4.3315	2972	2635	2303	1976	1653	1335	1022	0713	0408					
14	4.0108	9312	9520	9252	8947	8607	8291	8118	7848	7583					
15	3.7321	7082	6806	6554	6305	6059	5816	5576	5339	5105	Owing to the rapidity with which cotangent changes, mean differences are of no use.				
16	3.4879	4646	4420	4197	3977	3759	3544	3332	3122	2914					
17	3.2709	2506	2305	2106	1910	1716	1524	1334	1146	0961					
18	3.0777	0595	0415	0237	0061	9887	9714	9544	9375	9208					
19	2.9042	8878	8716	8550	8397	8239	8083	7929	7776	7625					
20	2.7475	7326	7179	7034	6889	6746	6605	6464	6325	6187	Owing to the rapidity with which cotangent changes, mean differences are of no use.				
21	2.6051	5916	5782	5649	5517	5386	5257	5129	5002	4876					
22	2.4751	4627	4504	4383	4262	4142	4023	3906	3789	3673					
23	2.3559	3445	3332	3220	3109	2998	2889	2781	2673	2566					
24	2.2460	2355	2251	2148	2045	1943	1842	1742	1642	1543					
25	2.1445	1348	1251	1155	1060	0965	0872	0778	0680	0594	Owing to the rapidity with which cotangent changes, mean differences are of no use.				
26	2.0503	0413	0323	0233	0145	0057	0970	9883	9797	9711					
27	1.9626	9542	9458	9375	9292	9210	9128	9047	8967	8887					
28	1.8807	8728	8650	8572	8495	8418	8341	8265	8190	8115					
29	1.8040	7696	7598	7500	7407	7315	7224	7132	7041	7391					
30	1.7321	7251	7182	7113	7045	6977	6909	6842	6775	6709	Owing to the rapidity with which cotangent changes, mean differences are of no use.				
31	1.6643	6577	6512	6447	6383	6319	6255	6191	6128	6066					
32	1.6008	5941	5880	5818	5757	5697	5637	5577	5517	5458					
33	1.5399	5340	5282	5224	5166	5108	5051	4994	4938	4882					
34	1.4826	4770	4715	4659	4605	4550	4496	4442	4388	4335					
35	1.4281	4229	4176	4124	4071	4019	3968	3916	3865	3814	Owing to the rapidity with which cotangent changes, mean differences are of no use.				
36	1.3764	3713	3663	3613	3564	3514	3465	3416	3367	3319					
37	1.3270	3222	3175	3127	3079	3032	2985	2938	2892	2846					
38	1.2799	2752	2708	2662	2617	2572	2527	2482	2437	2393					
39	1.2349	2306	2261	2218	2174	2131	2088	2045	2002	1960					
40	1.1918	1875	1833	1792	1750	1708	1667	1626	1586	1544	Owing to the rapidity with which cotangent changes, mean differences are of no use.				
41	1.1504	1463	1423	1383	1343	1303	1263	1224	1184	1145					
42	1.1106	1067	1028	0990	0951	0913	0875	0837	0799	0761					
43	1.0724	0686	0649	0612	0575	0538	0501	0464	0428	0392					
44	1.0355	0319	0283	0247	0212	0176	0141	0106	0070	0035					

# REDPATH, BROWN & CO., LIMITED.

## NATURAL COTANGENTS.

N.B.—Subtract Mean Differences.

	0'	6'	12'	18'	24'	30'	36'	42'	48'	54'	Mean Differences.				
											1'	2'	3'	4'	5'
45'	1.000	9965	9930	9896	9861	9827	9793	9759	9725	9691	6	11	17	23	29
46	.9657	9023	9590	9556	9523	9490	9457	9424	9391	9358	6	11	17	22	28
47	.9325	9293	9260	9228	9195	9163	9131	9099	9067	9035	5	11	16	21	27
48	.9004	8972	8941	8910	8878	8847	8816	8785	8754	8723	5	10	16	21	26
49	.8693	8662	8632	8601	8571	8541	8511	8481	8451	8421	5	10	15	20	25
50	.8391	8361	8331	8302	8273	8243	8214	8185	8156	8127	5	10	15	20	24
51	.8098	8069	8040	8012	7983	7954	7926	7898	7869	7841	5	10	14	19	24
52	.7813	7785	7757	7729	7701	7673	7646	7618	7590	7563	5	9	14	18	23
53	.7536	7509	7481	7454	7427	7400	7373	7346	7319	7292	5	9	14	18	23
54	.7265	7239	7212	7186	7159	7133	7107	7080	7054	7028	4	9	13	18	22
55	.7002	6976	6950	6924	6899	6873	6847	6822	6796	6771	4	9	13	17	21
56	.6745	6720	6694	6669	6644	6619	6594	6569	6544	6519	4	8	13	17	21
57	.6494	6469	6445	6420	6395	6371	6346	6322	6297	6273	4	8	12	16	20
58	.6249	6224	6200	6176	6152	6128	6104	6080	6056	6032	4	8	12	16	20
59	.6009	5985	5961	5938	5914	5890	5867	5844	5820	5797	4	8	12	16	20
60	.5774	5750	5727	5704	5681	5658	5635	5612	5589	5566	4	8	12	15	19
61	.5543	5520	5498	5475	5452	5430	5407	5384	5362	5340	4	8	11	15	19
62	.5317	5295	5272	5250	5228	5206	5184	5161	5139	5117	4	7	11	15	18
63	.5095	5073	5051	5029	5008	4986	4964	4942	4921	4899	4	7	11	15	18
64	.4877	4856	4834	4813	4791	4770	4748	4727	4706	4684	4	7	11	14	18
65	.4663	4642	4621	4600	4578	4557	4536	4515	4494	4473	4	7	10	14	18
66	.4462	4441	4421	4400	4379	4358	4337	4316	4296	4275	3	7	10	14	17
67	.4245	4224	4204	4183	4163	4142	4122	4101	4081	4061	3	7	10	14	17
68	.4040	4020	4000	3979	3959	3939	3919	3899	3879	3859	3	7	10	13	17
69	.3839	3819	3799	3779	3759	3739	3719	3699	3679	3659	3	7	10	13	17
70	.3640	3620	3600	3581	3561	3541	3522	3502	3482	3463	3	6	10	13	17
71	.3443	3424	3404	3385	3365	3346	3327	3307	3288	3269	3	6	10	13	16
72	.3249	3230	3211	3191	3172	3153	3134	3115	3096	3076	3	6	10	13	16
73	.3057	3038	3019	3000	2981	2962	2943	2924	2905	2886	3	6	9	13	16
74	.2867	2849	2830	2811	2792	2773	2754	2736	2717	2698	3	6	9	13	16
75	.2679	2661	2642	2623	2605	2586	2568	2549	2530	2512	3	6	9	12	16
76	.2493	2475	2456	2438	2419	2401	2382	2364	2345	2327	3	6	9	12	15
77	.2300	2280	2262	2244	2225	2207	2189	2170	2152	2134	3	6	9	12	15
78	.2126	2107	2089	2071	2053	2035	2016	1998	1980	1962	3	6	9	12	15
79	.1944	1926	1908	1890	1871	1853	1835	1817	1799	1781	3	6	9	12	15
80	.1763	1745	1727	1709	1691	1673	1655	1638	1620	1602	3	6	9	12	15
81	.1584	1566	1548	1530	1512	1495	1477	1459	1441	1423	3	6	9	12	15
82	.1405	1388	1370	1352	1334	1317	1299	1281	1263	1245	3	6	9	12	15
83	.1228	1210	1192	1175	1157	1139	1122	1104	1086	1069	3	6	9	12	15
84	.1051	1033	1016	0998	0981	0963	0945	0928	0910	0892	3	6	9	12	15
85	.0875	0857	0840	0822	0805	0787	0769	0752	0734	0717	3	6	9	12	15
86	.0699	0682	0664	0647	0629	0612	0594	0577	0559	0542	3	6	9	12	15
87	.0524	0507	0489	0472	0454	0437	0419	0402	0384	0367	3	6	9	12	15
88	.0349	0332	0314	0297	0279	0262	0244	0227	0209	0192	3	6	9	12	14
89	.0175	0157	0140	0122	0105	0087	0070	0052	0035	0017	3	6	9	12	15

# REDPATH, BROWN & CO., LIMITED.

## SQUARES.

	0	1	2	3	4	5	6	7	8	9	Mean Differences.								
											1	2	3	4	5	6	7	8	9
1'0	1'000	1'020	1'040	1'061	1'082	1'103	1'124	1'145	1'166	1'188	2	4	6	8	10	13	15	17	19
1'1	1'210	1'232	1'254	1'277	1'300	1'323	1'346	1'369	1'392	1'416	2	5	7	9	11	14	16	18	21
1'2	1'440	1'464	1'488	1'513	1'538	1'563	1'588	1'613	1'638	1'664	2	5	7	10	12	15	17	20	22
1'3	1'690	1'716	1'742	1'769	1'796	1'823	1'850	1'877	1'904	1'932	3	5	8	11	13	16	19	22	24
1'4	1'980	1'988	2'016	2'045	2'074	2'103	2'132	2'161	2'190	2'220	3	6	9	12	14	17	20	23	26
1'5	2'250	2'280	2'310	2'341	2'372	2'403	2'434	2'465	2'496	2'528	3	6	9	12	15	19	22	25	28
1'6	2'560	2'592	2'624	2'657	2'690	2'723	2'756	2'789	2'822	2'856	3	7	10	13	16	20	23	26	30
1'7	2'890	2'924	2'958	2'993	3'028	3'063	3'098	3'133	3'168	3'204	3	7	10	14	17	21	24	28	31
1'8	3'240	3'276	3'312	3'349	3'386	3'423	3'460	3'497	3'534	3'572	4	7	11	15	18	22	26	30	33
1'9	3'610	3'648	3'686	3'725	3'764	3'803	3'842	3'881	3'920	3'960	4	8	12	16	19	23	27	31	35
2'0	4'000	4'040	4'080	4'121	4'162	4'203	4'244	4'285	4'326	4'368	4	8	12	16	20	25	29	33	37
2'1	4'410	4'452	4'494	4'537	4'580	4'623	4'666	4'709	4'752	4'796	4	9	13	17	21	26	30	34	39
2'2	4'840	4'884	4'928	4'973	5'018	5'063	5'108	5'153	5'198	5'244	4	9	13	18	22	27	31	36	40
2'3	5'290	5'336	5'382	5'429	5'476	5'523	5'570	5'617	5'664	5'712	5	9	14	19	23	28	33	38	42
2'4	5'760	5'808	5'856	5'905	5'954	6'003	6'052	6'101	6'150	6'200	5	10	15	20	24	29	34	39	44
2'5	6'250	6'300	6'350	6'401	6'452	6'503	6'554	6'605	6'656	6'708	5	10	15	20	25	31	36	41	46
2'6	6'760	6'812	6'864	6'917	6'970	7'023	7'076	7'129	7'182	7'236	5	11	16	21	26	32	37	42	48
2'7	7'290	7'344	7'398	7'453	7'508	7'563	7'618	7'673	7'728	7'784	5	11	16	22	27	33	38	44	49
2'8	7'840	7'896	7'952	8'009	8'066	8'123	8'180	8'237	8'294	8'352	6	11	17	23	28	34	40	46	51
2'9	8'410	8'468	8'526	8'585	8'644	8'703	8'762	8'821	8'880	8'940	6	12	18	24	29	35	41	47	53
3'0	9'000	9'060	9'120	9'181	9'242	9'303	9'364	9'425	9'486	9'548	6	12	18	24	30	37	43	49	55
3'1	9'610	9'672	9'734	9'797	9'860	9'923	9'986				6	13	19	25	31	38	44	50	57
3'2	10'24	10'30	10'37	10'43	10'50	10'56	10'63	10'69	10'76	10'82	1	1	2	3	4	5	6	7	8
3'3	10'89	10'96	11'02	11'09	11'16	11'22	11'29	11'36	11'42	11'49	1	1	2	3	4	5	6	7	8
3'4	11'56	11'63	11'70	11'76	11'83	11'90	11'97	12'04	12'11	12'18	1	1	2	3	4	5	6	7	8
3'5	12'25	12'32	12'39	12'46	12'53	12'60	12'67	12'74	12'82	12'89	1	1	2	3	4	5	6	7	8
3'6	12'96	13'03	13'10	13'18	13'25	13'32	13'40	13'47	13'54	13'62	1	1	2	3	4	5	6	7	8
3'7	13'69	13'76	13'84	13'91	13'99	14'06	14'14	14'21	14'29	14'36	1	2	3	4	5	6	7	8	9
3'8	14'44	14'52	14'59	14'67	14'75	14'82	14'90	14'98	15'05	15'13	1	2	3	4	5	6	7	8	9
3'9	15'21	15'29	15'37	15'44	15'52	15'60	15'68	15'76	15'84	15'92	1	2	3	4	5	6	7	8	9
4'0	16'00	16'08	16'16	16'24	16'32	16'40	16'48	16'56	16'65	16'73	1	2	3	4	5	6	7	8	9
4'1	16'81	16'89	16'97	17'06	17'14	17'22	17'31	17'39	17'47	17'56	1	2	3	4	5	6	7	8	9
4'2	17'64	17'72	17'81	17'89	17'98	18'06	18'15	18'23	18'32	18'40	1	2	3	4	5	6	7	8	9
4'3	18'49	18'58	18'66	18'75	18'84	18'92	19'01	19'10	19'18	19'27	1	2	3	4	5	6	7	8	9
4'4	19'36	19'45	19'54	19'62	19'71	19'80	19'89	19'98	20'07	20'16	1	2	3	4	5	6	7	8	9
4'5	20'25	20'34	20'43	20'52	20'61	20'70	20'79	20'88	20'98	21'07	1	2	3	4	5	6	7	8	9
4'6	21'16	21'25	21'34	21'44	21'53	21'62	21'72	21'81	21'90	22'00	1	2	3	4	5	6	7	8	9
4'7	22'09	22'18	22'28	22'37	22'47	22'56	22'66	22'75	22'85	22'94	1	2	3	4	5	6	7	8	9
4'8	23'04	23'14	23'23	23'33	23'43	23'52	23'62	23'72	23'81	23'91	1	2	3	4	5	6	7	8	9
4'9	24'01	24'11	24'21	24'30	24'40	24'50	24'60	24'70	24'80	24'90	1	2	3	4	5	6	7	8	9
5'0	25'00	25'10	25'20	25'30	25'40	25'50	25'60	25'70	25'81	25'91	1	2	3	4	5	6	7	8	9
5'1	26'00	26'11	26'21	26'32	26'42	26'52	26'63	26'73	26'84	26'94	1	2	3	4	5	6	7	8	9
5'2	27'04	27'14	27'25	27'35	27'46	27'56	27'67	27'77	27'88	27'98	1	2	3	4	5	6	7	8	9
5'3	28'09	28'20	28'30	28'41	28'52	28'62	28'73	28'84	28'94	29'05	1	2	3	4	5	6	7	8	9
5'4	29'16	29'27	29'38	29'48	29'59	30'09	30'19	30'29	30'39	30'49	1	2	3	4	5	6	7	8	9

# REDPATH, BROWN & CO., LIMITED.

## SQUARES.

	0	1	2	3	4	5	6	7	8	9	Mean Differences.											
											1	2	3	4	5	6	7	8	9			
5-5	30-25	30-36	30-47	30-58	30-69	30-80	30-91	31-02	31-14	31-25	1	2	3	4	5	6	7	8	9	10	11	12
5-6	31-36	31-47	31-58	31-70	31-81	31-92	32-04	32-15	32-26	32-38	1	2	3	4	5	6	7	8	9	10	11	12
5-7	32-49	32-60	32-72	32-83	32-95	33-06	33-18	33-29	33-41	33-52	1	2	3	4	5	6	7	8	9	10	11	12
5-8	33-64	33-76	33-87	33-99	34-11	34-22	34-34	34-46	34-57	34-69	1	2	3	4	5	6	7	8	9	10	11	12
5-9	34-81	34-93	35-05	35-16	35-28	35-40	35-52	35-64	35-76	35-88	1	2	3	4	5	6	7	8	9	10	11	12
6-0	36-00	36-12	36-24	36-36	36-48	36-60	36-72	36-84	36-97	37-09	1	2	3	4	5	6	7	8	9	10	11	12
6-1	37-21	37-33	37-45	37-57	37-70	37-82	37-95	38-07	38-19	38-32	1	2	3	4	5	6	7	8	9	10	11	12
6-2	38-44	38-56	38-69	38-81	38-94	39-06	39-19	39-31	39-44	39-56	1	2	3	4	5	6	7	8	9	10	11	12
6-3	39-69	39-82	39-94	40-07	40-20	40-32	40-45	40-58	40-70	40-83	1	2	3	4	5	6	7	8	9	10	11	12
6-4	40-96	41-09	41-22	41-34	41-47	41-60	41-73	41-86	41-99	42-12	1	2	3	4	5	6	7	8	9	10	11	12
6-5	42-25	42-38	42-51	42-64	42-77	42-90	43-02	43-16	43-30	43-43	1	2	3	4	5	6	7	8	9	10	11	12
6-6	43-56	43-69	43-82	43-96	44-09	44-22	44-35	44-49	44-62	44-76	1	2	3	4	5	6	7	8	9	10	11	12
6-7	44-89	45-02	45-16	45-29	45-43	45-56	45-70	45-83	45-97	46-10	1	2	3	4	5	6	7	8	9	10	11	12
6-8	46-24	46-38	46-51	46-65	46-79	46-92	47-06	47-20	47-33	47-47	1	2	3	4	5	6	7	8	9	10	11	12
6-9	47-61	47-75	47-89	48-03	48-16	48-30	48-44	48-58	48-72	48-86	1	2	3	4	5	6	7	8	9	10	11	12
7-0	49-00	49-14	49-28	49-42	49-56	49-70	49-84	49-98	50-13	50-27	1	2	3	4	5	6	7	8	9	10	11	12
7-1	50-41	50-55	50-69	50-84	50-98	51-12	51-27	51-41	51-55	51-70	1	2	3	4	5	6	7	8	9	10	11	12
7-2	51-84	51-98	52-13	52-27	52-42	52-56	52-71	52-85	53-00	53-14	1	2	3	4	5	6	7	8	9	10	11	12
7-3	53-29	53-44	53-58	53-73	53-88	54-02	54-17	54-32	54-46	54-61	1	2	3	4	5	6	7	8	9	10	11	12
7-4	54-76	54-91	55-06	55-20	55-35	55-50	55-65	55-80	55-95	56-10	1	2	3	4	5	6	7	8	9	10	11	12
7-5	56-25	56-40	56-55	56-70	56-85	57-00	57-15	57-30	57-46	57-61	2	3	4	5	6	7	8	9	10	11	12	13
7-6	57-76	57-91	58-06	58-22	58-37	58-52	58-68	58-83	58-98	59-14	2	3	4	5	6	7	8	9	10	11	12	13
7-7	59-29	59-44	59-60	59-75	59-91	60-06	60-22	60-37	60-53	60-68	2	3	4	5	6	7	8	9	10	11	12	13
7-8	60-84	61-00	61-15	61-31	61-47	61-62	61-78	61-94	62-09	62-25	2	3	4	5	6	7	8	9	10	11	12	13
7-9	62-41	62-57	62-73	62-88	63-04	63-20	63-36	63-52	63-68	63-84	2	3	4	5	6	7	8	9	10	11	12	13
8-0	64-00	64-16	64-32	64-48	64-64	64-80	64-96	65-12	65-29	65-45	2	3	4	5	6	7	8	9	10	11	12	13
8-1	65-61	65-77	65-93	66-10	66-26	66-42	66-59	66-75	66-91	67-08	2	3	4	5	6	7	8	9	10	11	12	13
8-2	67-24	67-40	67-57	67-73	67-90	68-06	68-23	68-39	68-56	68-72	2	3	4	5	6	7	8	9	10	11	12	13
8-3	68-89	69-06	69-22	69-39	69-56	69-72	69-89	70-06	70-22	70-39	2	3	4	5	6	7	8	9	10	11	12	13
8-4	70-56	70-73	70-90	71-06	71-23	71-40	71-57	71-74	71-91	72-08	2	3	4	5	6	7	8	9	10	11	12	13
8-5	72-25	72-42	72-59	72-76	72-93	73-10	73-27	73-44	73-62	73-79	2	3	4	5	6	7	8	9	10	11	12	13
8-6	73-96	74-13	74-30	74-48	74-65	74-82	75-00	75-17	75-34	75-52	2	3	4	5	6	7	8	9	10	11	12	13
8-7	75-69	75-86	76-04	76-21	76-39	76-56	76-74	76-91	77-09	77-26	2	3	4	5	6	7	8	9	10	11	12	13
8-8	77-44	77-62	77-80	77-97	78-15	78-32	78-50	78-68	78-85	79-03	2	3	4	5	6	7	8	9	10	11	12	13
8-9	79-21	79-39	79-57	79-74	79-92	80-10	80-28	80-46	80-64	80-82	2	3	4	5	6	7	8	9	10	11	12	13
9-0	81-00	81-18	81-36	81-54	81-72	81-90	82-08	82-26	82-45	82-63	2	3	4	5	6	7	8	9	10	11	12	13
9-1	82-81	82-99	83-17	83-36	83-54	83-72	83-91	84-09	84-27	84-46	2	3	4	5	6	7	8	9	10	11	12	13
9-2	84-64	84-82	85-01	85-19	85-38	85-56	85-75	85-93	86-12	86-30	2	3	4	5	6	7	8	9	10	11	12	13
9-3	86-49	86-68	86-86	87-05	87-24	87-42	87-61	87-80	87-98	88-17	2	3	4	5	6	7	8	9	10	11	12	13
9-4	88-36	88-55	88-74	88-92	89-11	89-30	89-49	89-68	89-87	90-06	2	3	4	5	6	7	8	9	10	11	12	13
9-5	90-25	90-44	90-63	90-82	91-01	91-20	91-39	91-58	91-78	91-97	2	3	4	5	6	7	8	9	10	11	12	13
9-6	92-16	92-35	92-54	92-74	92-93	93-12	93-32	93-51	93-70	93-90	2	3	4	5	6	7	8	9	10	11	12	13
9-7	94-09	94-28	94-48	94-67	94-87	95-06	95-26	95-45	95-65	95-84	2	3	4	5	6	7	8	9	10	11	12	13
9-8	96-04	96-24	96-43	96-63	96-83	97-02	97-22	97-42	97-61	97-81	2	3	4	5	6	7	8	9	10	11	12	13
9-9	98-01	98-21	98-41	98-60	98-80	99-00	99-20	99-40	99-60	99-80	2	3	4	5	6	7	8	9	10	11	12	13

# REDPATH, BROWN & CO., LIMITED.

## SQUARE ROOTS FROM 100 TO 999.9.

	0	1	2	3	4	5	6	7	8	9	Mean Differences.								
											1	2	3	4	5	6	7	8	9
10	10.00	10.05	10.10	10.15	10.20	10.25	10.30	10.34	10.39	10.44	0	1	1	2	2	3	3	4	4
11	10.49	10.54	10.58	10.63	10.68	10.72	10.77	10.82	10.86	10.91	0	1	1	2	2	3	3	4	4
12	10.95	11.00	11.05	11.09	11.14	11.18	11.22	11.27	11.31	11.36	0	1	1	2	2	3	3	4	4
13	11.40	11.45	11.49	11.53	11.58	11.62	11.66	11.70	11.75	11.79	0	1	1	2	2	3	3	4	4
14	11.83	11.87	11.92	11.96	12.00	12.04	12.08	12.12	12.17	12.21	0	1	1	2	2	3	3	4	4
15	12.25	12.29	12.33	12.37	12.41	12.45	12.49	12.53	12.57	12.61	0	1	1	2	2	3	3	4	4
16	12.65	12.69	12.73	12.77	12.81	12.85	12.88	12.92	12.96	13.00	0	1	1	2	2	3	3	4	4
17	13.04	13.08	13.11	13.15	13.19	13.23	13.27	13.30	13.34	13.38	0	1	1	2	2	3	3	4	4
18	13.42	13.45	13.49	13.53	13.56	13.60	13.64	13.67	13.71	13.75	0	1	1	2	2	3	3	4	4
19	13.78	13.82	13.86	13.89	13.93	13.96	14.00	14.04	14.07	14.11	0	1	1	2	2	3	3	4	4
20	14.14	14.18	14.21	14.25	14.28	14.32	14.35	14.39	14.42	14.46	0	1	1	2	2	3	3	4	4
21	14.49	14.53	14.56	14.59	14.63	14.66	14.70	14.73	14.76	14.80	0	1	1	2	2	3	3	4	4
22	14.83	14.87	14.90	14.93	14.97	15.00	15.03	15.07	15.10	15.13	0	1	1	2	2	3	3	4	4
23	15.17	15.20	15.23	15.26	15.30	15.33	15.36	15.39	15.43	15.46	0	1	1	2	2	3	3	4	4
24	15.49	15.52	15.56	15.59	15.62	15.65	15.68	15.72	15.75	15.78	0	1	1	2	2	3	3	4	4
25	15.81	15.84	15.87	15.91	15.94	15.97	16.00	16.03	16.06	16.09	0	1	1	2	2	3	3	4	4
26	16.12	16.16	16.19	16.22	16.25	16.28	16.31	16.34	16.37	16.40	0	1	1	2	2	3	3	4	4
27	16.43	16.46	16.49	16.52	16.55	16.58	16.61	16.64	16.67	16.70	0	1	1	2	2	3	3	4	4
28	16.73	16.76	16.79	16.82	16.85	16.88	16.91	16.94	16.97	17.00	0	1	1	2	2	3	3	4	4
29	17.03	17.06	17.09	17.12	17.15	17.18	17.20	17.23	17.26	17.29	0	1	1	2	2	3	3	4	4
30	17.32	17.35	17.38	17.41	17.44	17.46	17.49	17.52	17.55	17.58	0	1	1	2	2	3	3	4	4
31	17.61	17.64	17.66	17.69	17.72	17.75	17.78	17.80	17.83	17.86	0	1	1	2	2	3	3	4	4
32	17.89	17.92	17.94	17.97	18.00	18.03	18.06	18.08	18.11	18.14	0	1	1	2	2	3	3	4	4
33	18.17	18.19	18.22	18.25	18.28	18.30	18.33	18.36	18.38	18.41	0	1	1	2	2	3	3	4	4
34	18.44	18.47	18.49	18.52	18.55	18.57	18.60	18.63	18.65	18.68	0	1	1	2	2	3	3	4	4
35	18.71	18.73	18.76	18.79	18.81	18.84	18.87	18.89	18.92	18.95	0	1	1	2	2	3	3	4	4
36	18.97	19.00	19.03	19.05	19.08	19.10	19.13	19.16	19.18	19.21	0	1	1	2	2	3	3	4	4
37	19.24	19.26	19.29	19.31	19.34	19.36	19.39	19.42	19.44	19.47	0	1	1	2	2	3	3	4	4
38	19.49	19.52	19.54	19.57	19.60	19.62	19.65	19.67	19.70	19.72	0	1	1	2	2	3	3	4	4
39	19.75	19.77	19.80	19.82	19.85	19.87	19.90	19.92	19.95	19.97	0	1	1	2	2	3	3	4	4
40	20.00	20.02	20.05	20.07	20.10	20.12	20.15	20.17	20.20	20.22	0	0	1	1	1	1	2	2	2
41	20.25	20.27	20.30	20.32	20.35	20.37	20.40	20.42	20.45	20.47	0	0	1	1	1	1	2	2	2
42	20.49	20.52	20.54	20.57	20.59	20.62	20.64	20.66	20.69	20.71	0	0	1	1	1	1	2	2	2
43	20.74	20.76	20.78	20.81	20.83	20.86	20.88	20.90	20.93	20.95	0	0	1	1	1	1	2	2	2
44	20.98	21.00	21.02	21.05	21.07	21.10	21.12	21.14	21.17	21.19	0	0	1	1	1	1	2	2	2
45	21.21	21.24	21.26	21.28	21.31	21.33	21.35	21.38	21.40	21.42	0	0	1	1	1	1	2	2	2
46	21.45	21.47	21.49	21.52	21.54	21.56	21.59	21.61	21.63	21.66	0	0	1	1	1	1	2	2	2
47	21.68	21.70	21.73	21.75	21.77	21.79	21.82	21.84	21.86	21.89	0	0	1	1	1	1	2	2	2
48	21.91	21.93	21.95	21.98	22.00	22.02	22.05	22.07	22.09	22.11	0	0	1	1	1	1	2	2	2
49	22.14	22.16	22.18	22.20	22.23	22.25	22.27	22.29	22.32	22.34	0	0	1	1	1	1	2	2	2
50	22.36	22.38	22.41	22.43	22.45	22.47	22.49	22.52	22.54	22.56	0	0	1	1	1	1	2	2	2
51	22.58	22.61	22.63	22.65	22.67	22.69	22.72	22.74	22.76	22.78	0	0	1	1	1	1	2	2	2
52	22.80	22.83	22.85	22.87	22.89	22.91	22.93	22.96	22.98	23.00	0	0	1	1	1	1	2	2	2
53	23.02	23.04	23.07	23.09	23.11	23.13	23.15	23.17	23.19	23.22	0	0	1	1	1	1	2	2	2
54	23.24	23.26	23.28	23.30	23.32	23.35	23.37	23.39	23.41	23.43	0	0	1	1	1	1	2	2	2

# REDPATH, BROWN & CO., LIMITED.

## SQUARE ROOTS FROM 100 TO 999.9.

	0	1	2	3	4	5	6	7	8	9	Mean Differences.								
											1	2	3	4	5	6	7	8	9
55	23.45	23.47	23.49	23.52	23.54	23.56	23.58	23.60	23.62	23.64	0	0	1	1	1	1	1	2	2
56	23.66	23.69	23.71	23.73	23.75	23.77	23.79	23.81	23.83	23.86	0	0	1	1	1	1	1	2	2
57	23.87	23.90	23.92	23.94	23.96	23.98	24.00	24.02	24.04	24.06	0	0	1	1	1	1	1	2	2
58	24.08	24.10	24.12	24.15	24.17	24.19	24.21	24.23	24.25	24.27	0	0	1	1	1	1	1	2	2
59	24.29	24.31	24.33	24.35	24.37	24.39	24.41	24.43	24.45	24.47	0	0	1	1	1	1	1	2	2
60	24.49	24.52	24.54	24.56	24.58	24.60	24.62	24.64	24.66	24.68	0	0	1	1	1	1	1	2	2
61	24.70	24.72	24.74	24.76	24.78	24.80	24.82	24.84	24.86	24.88	0	0	1	1	1	1	1	2	2
62	24.90	24.92	24.94	24.96	24.98	25.00	25.02	25.04	25.06	25.08	0	0	1	1	1	1	1	2	2
63	25.10	25.12	25.14	25.16	25.18	25.20	25.22	25.24	25.26	25.28	0	0	1	1	1	1	1	2	2
64	25.30	25.32	25.34	25.36	25.38	25.40	25.42	25.44	25.46	25.48	0	0	1	1	1	1	1	2	2
65	25.50	25.51	25.53	25.55	25.57	25.59	25.61	25.63	25.65	25.67	0	0	1	1	1	1	1	2	2
66	25.69	25.71	25.73	25.75	25.77	25.79	25.81	25.83	25.85	25.87	0	0	1	1	1	1	1	2	2
67	25.88	25.90	25.92	25.94	25.96	25.98	26.00	26.02	26.04	26.06	0	0	1	1	1	1	1	2	2
68	26.08	26.10	26.12	26.13	26.15	26.17	26.19	26.21	26.23	26.25	0	0	1	1	1	1	1	2	2
69	26.27	26.29	26.31	26.32	26.34	26.36	26.38	26.40	26.42	26.44	0	0	1	1	1	1	1	2	2
70	26.46	26.48	26.50	26.51	26.53	26.55	26.57	26.59	26.61	26.63	0	0	1	1	1	1	1	2	2
71	26.66	26.68	26.69	26.70	26.72	26.74	26.76	26.78	26.80	26.81	0	0	1	1	1	1	1	2	2
72	26.83	26.85	26.87	26.89	26.91	26.93	26.94	26.96	26.98	27.00	0	0	1	1	1	1	1	2	2
73	27.02	27.04	27.06	27.07	27.09	27.11	27.13	27.15	27.17	27.18	0	0	1	1	1	1	1	2	2
74	27.20	27.22	27.24	27.26	27.28	27.29	27.31	27.33	27.35	27.37	0	0	1	1	1	1	1	2	2
75	27.39	27.40	27.42	27.44	27.46	27.48	27.50	27.51	27.53	27.55	0	0	1	1	1	1	1	2	2
76	27.57	27.59	27.60	27.62	27.64	27.66	27.68	27.69	27.71	27.73	0	0	1	1	1	1	1	2	2
77	27.75	27.77	27.78	27.80	27.82	27.84	27.86	27.87	27.89	27.91	0	0	1	1	1	1	1	2	2
78	27.93	27.95	27.96	27.98	28.00	28.02	28.04	28.06	28.07	28.09	0	0	1	1	1	1	1	2	2
79	28.11	28.12	28.14	28.16	28.18	28.20	28.21	28.23	28.25	28.27	0	0	1	1	1	1	1	2	2
80	28.28	28.30	28.32	28.34	28.35	28.37	28.39	28.41	28.43	28.44	0	0	1	1	1	1	1	2	2
81	28.46	28.48	28.50	28.51	28.53	28.55	28.57	28.58	28.60	28.62	0	0	1	1	1	1	1	2	2
82	28.64	28.66	28.67	28.69	28.71	28.72	28.74	28.76	28.77	28.79	0	0	1	1	1	1	1	2	2
83	28.81	28.83	28.84	28.86	28.88	28.90	28.91	28.93	28.95	28.97	0	0	1	1	1	1	1	2	2
84	28.98	29.00	29.02	29.03	29.05	29.07	29.09	29.10	29.12	29.14	0	0	1	1	1	1	1	2	2
85	29.15	29.17	29.19	29.21	29.22	29.24	29.26	29.27	29.29	29.31	0	0	1	1	1	1	1	2	2
86	29.33	29.34	29.36	29.38	29.39	29.41	29.43	29.44	29.46	29.48	0	0	1	1	1	1	1	2	2
87	29.50	29.51	29.53	29.55	29.56	29.58	29.60	29.61	29.63	29.65	0	0	1	1	1	1	1	2	2
88	29.66	29.68	29.70	29.72	29.73	29.75	29.77	29.78	29.80	29.82	0	0	1	1	1	1	1	2	2
89	29.83	29.85	29.87	29.88	29.90	29.92	29.93	29.95	29.97	29.98	0	0	1	1	1	1	1	2	2
90	30.00	30.02	30.03	30.06	30.07	30.08	30.10	30.12	30.13	30.15	0	0	0	1	1	1	1	1	1
91	30.17	30.18	30.20	30.22	30.23	30.25	30.27	30.28	30.30	30.32	0	0	0	1	1	1	1	1	1
92	30.33	30.35	30.36	30.38	30.40	30.41	30.43	30.45	30.46	30.48	0	0	0	1	1	1	1	1	1
93	30.50	30.51	30.53	30.55	30.56	30.58	30.59	30.61	30.63	30.64	0	0	0	1	1	1	1	1	1
94	30.66	30.68	30.69	30.71	30.72	30.74	30.76	30.77	30.79	30.81	0	0	0	1	1	1	1	1	1
95	30.82	30.84	30.85	30.87	30.89	30.90	30.92	30.94	30.96	30.97	0	0	0	1	1	1	1	1	1
96	30.98	31.00	31.02	31.03	31.05	31.06	31.08	31.10	31.11	31.13	0	0	0	1	1	1	1	1	1
97	31.14	31.16	31.18	31.19	31.21	31.22	31.24	31.26	31.27	31.29	0	0	0	1	1	1	1	1	1
98	31.30	31.32	31.34	31.36	31.37	31.38	31.40	31.42	31.43	31.45	0	0	0	1	1	1	1	1	1
99	31.46	31.48	31.50	31.51	31.53	31.54	31.56	31.58	31.59	31.61	0	0	0	1	1	1	1	1	1

# REDPATH, BROWN & CO., LIMITED.

## SQUARE ROOTS FROM 1000 TO 9999.

	0	1	2	3	4	5	6	7	8	9	Mean Differences.								
											1	2	3	4	5	6	7	8	9
10	31.62	31.78	31.94	32.09	32.25	32.40	32.56	32.71	32.86	33.02	2	3	5	6	8	9	11	12	14
11	33.17	33.32	33.47	33.62	33.76	33.91	34.06	34.21	34.35	34.50	1	3	4	6	7	9	10	12	13
12	34.64	34.79	34.93	35.07	35.21	35.36	35.50	35.64	35.78	35.92	1	3	4	6	7	8	10	11	13
13	36.06	36.19	36.33	36.47	36.61	36.74	36.88	37.01	37.15	37.28	1	3	4	5	7	8	10	11	12
14	37.42	37.55	37.68	37.82	37.95	38.08	38.21	38.34	38.47	38.60	1	3	4	5	7	8	9	11	12
15	38.73	38.86	38.99	39.12	39.24	39.37	39.50	39.62	39.75	39.87	1	3	4	5	6	8	9	10	11
16	40.00	40.12	40.25	40.37	40.50	40.62	40.74	40.87	40.99	41.11	1	2	4	5	6	7	9	10	11
17	41.23	41.35	41.47	41.59	41.71	41.83	41.95	42.07	42.19	42.31	1	2	4	5	6	7	8	10	11
18	42.43	42.54	42.66	42.78	42.90	43.01	43.13	43.24	43.36	43.47	1	2	3	5	6	7	8	9	10
19	43.59	43.70	43.82	43.93	44.05	44.16	44.27	44.38	44.50	44.61	1	2	3	5	6	7	8	9	10
20	44.72	44.83	44.94	45.05	45.17	45.28	45.39	45.50	45.61	45.72	1	2	3	4	6	7	8	9	10
21	45.83	45.93	46.04	46.15	46.26	46.37	46.48	46.58	46.69	46.80	1	2	3	4	5	6	8	9	10
22	46.90	47.01	47.12	47.22	47.33	47.43	47.54	47.64	47.75	47.85	1	2	3	4	5	6	7	8	9
23	47.96	48.06	48.17	48.27	48.37	48.48	48.58	48.68	48.79	48.89	1	2	3	4	5	6	7	8	9
24	48.99	49.09	49.19	49.30	49.40	49.50	49.60	49.70	49.80	49.90	1	2	3	4	5	6	7	8	9
25	50.00	50.10	50.20	50.30	50.40	50.50	50.60	50.70	50.79	50.89	1	2	3	4	5	6	7	8	9
26	50.99	51.09	51.19	51.28	51.38	51.48	51.58	51.67	51.77	51.87	1	2	3	4	5	6	7	8	9
27	51.96	52.06	52.15	52.25	52.35	52.44	52.54	52.63	52.73	52.82	1	2	3	4	5	6	7	8	9
28	52.92	53.01	53.10	53.20	53.29	53.39	53.48	53.57	53.67	53.76	1	2	3	4	5	6	7	8	9
29	53.85	53.94	54.04	54.13	54.22	54.31	54.41	54.50	54.59	54.68	1	2	3	4	5	6	7	8	9
30	54.77	54.86	54.95	55.05	55.14	55.23	55.32	55.41	55.50	55.59	1	2	3	4	5	6	7	8	9
31	55.68	55.77	55.86	55.95	56.04	56.12	56.21	56.30	56.39	56.48	1	2	3	3	4	5	6	7	8
32	56.57	56.66	56.75	56.83	56.92	57.01	57.10	57.18	57.27	57.36	1	2	3	3	4	5	6	7	8
33	57.45	57.53	57.62	57.71	57.79	57.88	57.97	58.05	58.14	58.22	1	2	3	3	4	5	6	7	8
34	58.31	58.40	58.48	58.57	58.65	58.74	58.82	58.91	58.99	59.08	1	2	3	3	4	5	6	7	8
35	59.16	59.25	59.33	59.41	59.50	59.58	59.67	59.75	59.83	59.92	1	2	2	3	4	5	6	7	8
36	60.00	60.08	60.17	60.25	60.33	60.42	60.50	60.58	60.66	60.75	1	2	2	3	4	5	6	7	7
37	60.83	60.91	60.99	61.07	61.16	61.24	61.32	61.40	61.48	61.56	1	2	2	3	4	5	6	7	7
38	61.64	61.73	61.81	61.89	61.97	62.05	62.13	62.21	62.29	62.37	1	2	2	3	4	5	6	7	7
39	62.45	62.53	62.61	62.69	62.77	62.85	62.93	63.01	63.09	63.17	1	2	2	3	4	5	6	6	7
40	63.25	63.32	63.40	63.48	63.56	63.64	63.72	63.80	63.87	63.95	1	2	2	3	4	5	6	6	7
41	64.03	64.11	64.19	64.27	64.34	64.42	64.50	64.58	64.65	64.73	1	2	2	3	4	5	6	6	7
42	64.81	64.88	64.96	65.04	65.12	65.19	65.27	65.35	65.42	65.50	1	2	2	3	4	5	6	6	7
43	65.58	65.65	65.73	65.80	65.88	65.95	66.03	66.11	66.18	66.26	1	2	2	3	4	5	6	6	7
44	66.33	66.41	66.48	66.56	66.63	66.71	66.78	66.86	66.93	67.01	1	2	2	3	4	5	6	6	7
45	67.08	67.16	67.23	67.31	67.38	67.45	67.53	67.60	67.68	67.75	1	1	2	3	4	4	5	6	7
46	67.82	67.90	67.97	68.04	68.12	68.19	68.26	68.34	68.41	68.48	1	1	2	3	4	4	5	6	7
47	68.56	68.63	68.70	68.77	68.85	68.92	68.99	69.07	69.14	69.21	1	1	2	3	4	4	5	6	7
48	69.28	69.35	69.43	69.50	69.57	69.64	69.71	69.78	69.86	69.93	1	1	2	3	4	4	5	6	6
49	70.00	70.07	70.14	70.21	70.29	70.36	70.43	70.50	70.57	70.64	1	1	2	3	4	4	5	6	6
50	70.71	70.78	70.85	70.92	70.99	71.06	71.13	71.20	71.27	71.34	1	1	2	3	4	4	5	6	6
51	71.41	71.48	71.55	71.62	71.69	71.76	71.83	71.90	71.97	72.04	1	1	2	3	4	4	5	6	6
52	72.11	72.18	72.25	72.32	72.39	72.46	72.53	72.60	72.66	72.73	1	1	2	3	4	4	5	6	6
53	72.80	72.87	72.94	73.01	73.08	73.14	73.21	73.28	73.35	73.42	1	1	2	3	4	4	5	6	6
54	73.48	73.55	73.62	73.69	73.76	73.82	73.89	73.96	74.03	74.09	1	1	2	3	4	4	5	6	6

# REDPATH, BROWN & CO., LIMITED.

## SQUARE ROOTS FROM 1000 TO 9999.

	0	1	2	3	4	5	6	7	8	9	Mean Differences.								
											1	2	3	4	5	6	7	8	9
55	74.16	74.23	74.30	74.36	74.43	74.50	74.57	74.63	74.70	74.77	1	1	2	3	3	4	5	5	6
56	74.83	74.90	74.97	75.03	75.10	75.17	75.23	75.30	75.37	75.43	1	1	2	3	3	4	5	5	6
57	75.50	75.56	75.63	75.70	75.76	75.83	75.89	75.96	76.03	76.09	1	1	2	3	3	4	5	5	6
58	76.16	76.22	76.29	76.35	76.42	76.49	76.55	76.62	76.68	76.75	1	1	2	3	3	4	5	5	6
59	76.81	76.88	76.94	77.01	77.07	77.14	77.20	77.27	77.33	77.40	1	1	2	3	3	4	4	5	6
60	77.46	77.52	77.59	77.65	77.72	77.78	77.85	77.91	77.97	78.04	1	1	2	3	3	4	4	5	6
61	78.10	78.17	78.23	78.29	78.36	78.42	78.49	78.55	78.61	78.68	1	1	2	3	3	4	4	5	6
62	78.74	78.80	78.87	78.93	78.99	79.06	79.12	79.18	79.25	79.31	1	1	2	3	3	4	4	5	6
63	79.37	79.44	79.50	79.56	79.62	79.69	79.75	79.81	79.87	79.94	1	1	2	3	3	4	4	5	6
64	80.00	80.06	80.12	80.19	80.25	80.31	80.37	80.44	80.50	80.56	1	1	2	2	3	4	4	5	6
65	80.62	80.68	80.75	80.81	80.87	80.93	80.99	81.06	81.12	81.18	1	1	2	2	3	4	4	5	5
66	81.24	81.30	81.36	81.42	81.49	81.55	81.61	81.67	81.73	81.79	1	1	2	2	3	4	4	5	5
67	81.85	81.91	81.98	82.04	82.10	82.16	82.22	82.28	82.34	82.40	1	1	2	2	3	4	4	5	5
68	82.46	82.52	82.58	82.64	82.70	82.76	82.83	82.89	82.95	83.01	1	1	2	2	3	4	4	5	5
69	83.07	83.13	83.19	83.25	83.31	83.37	83.43	83.49	83.55	83.61	1	1	2	2	3	4	4	5	5
70	83.67	83.73	83.79	83.85	83.90	83.96	84.02	84.08	84.14	84.20	1	1	2	2	3	4	4	5	5
71	84.26	84.32	84.38	84.44	84.50	84.56	84.62	84.68	84.73	84.79	1	1	2	2	3	4	4	5	5
72	84.85	84.91	84.97	85.03	85.09	85.15	85.21	85.26	85.32	85.38	1	1	2	2	3	4	4	5	5
73	85.44	85.50	85.56	85.62	85.67	85.73	85.79	85.85	85.91	85.97	1	1	2	2	3	4	4	5	5
74	86.02	86.08	86.14	86.20	86.26	86.31	86.37	86.43	86.49	86.54	1	1	2	2	3	4	4	5	5
75	86.60	86.66	86.72	86.78	86.83	86.89	86.95	87.01	87.06	87.12	1	1	2	2	3	4	4	5	5
76	87.18	87.24	87.29	87.35	87.41	87.46	87.52	87.58	87.64	87.69	1	1	2	2	3	4	4	5	5
77	87.75	87.81	87.86	87.92	87.98	88.03	88.09	88.15	88.20	88.26	1	1	2	2	3	4	4	5	5
78	88.32	88.37	88.43	88.49	88.54	88.60	88.66	88.71	88.77	88.83	1	1	2	2	3	4	4	5	5
79	88.88	88.94	88.99	89.05	89.11	89.16	89.22	89.27	89.33	89.39	1	1	2	2	3	4	4	5	5
80	89.44	89.50	89.55	89.61	89.67	89.72	89.78	89.83	89.89	89.94	1	1	2	2	3	4	4	5	5
81	90.00	90.06	90.11	90.17	90.22	90.28	90.33	90.39	90.44	90.50	1	1	2	2	3	4	4	5	5
82	90.55	90.61	90.66	90.72	90.77	90.83	90.88	90.94	90.99	91.05	1	1	2	2	3	4	4	5	5
83	91.10	91.16	91.21	91.27	91.32	91.38	91.43	91.49	91.54	91.60	1	1	2	2	3	4	4	5	5
84	91.65	91.71	91.76	91.82	91.87	91.92	91.98	92.03	92.09	92.14	1	1	2	2	3	4	4	5	5
85	92.20	92.25	92.30	92.36	92.41	92.47	92.52	92.57	92.63	92.68	1	1	2	2	3	4	4	5	5
86	92.74	92.79	92.84	92.90	92.95	93.01	93.06	93.11	93.17	93.22	1	1	2	2	3	4	4	5	5
87	93.27	93.33	93.38	93.43	93.49	93.54	93.59	93.65	93.70	93.75	1	1	2	2	3	4	4	5	5
88	93.81	93.86	93.91	93.97	94.02	94.07	94.13	94.18	94.23	94.29	1	1	2	2	3	4	4	5	5
89	94.34	94.39	94.45	94.50	94.55	94.60	94.66	94.71	94.76	94.82	1	1	2	2	3	4	4	5	5
90	94.87	94.92	94.97	95.03	95.08	95.13	95.18	95.24	95.29	95.34	1	1	2	2	3	4	4	5	5
91	95.39	95.45	95.50	95.55	95.60	95.66	95.71	95.76	95.81	95.86	1	1	2	2	3	4	4	5	5
92	95.92	95.97	96.02	96.07	96.12	96.18	96.23	96.28	96.33	96.38	1	1	2	2	3	4	4	5	5
93	96.44	96.49	96.54	96.59	96.64	96.70	96.75	96.80	96.85	96.90	1	1	2	2	3	4	4	5	5
94	96.95	97.01	97.06	97.11	97.16	97.21	97.26	97.31	97.37	97.42	1	1	2	2	3	4	4	5	5
95	97.47	97.52	97.57	97.62	97.67	97.72	97.78	97.83	97.88	97.93	1	1	2	2	3	4	4	5	5
96	97.98	98.03	98.08	98.13	98.18	98.23	98.29	98.34	98.39	98.44	1	1	2	2	3	4	4	5	5
97	98.49	98.54	98.59	98.64	98.69	98.74	98.79	98.84	98.89	98.94	1	1	2	2	3	4	4	5	5
98	98.99	99.05	99.10	99.15	99.20	99.25	99.30	99.35	99.40	99.45	0	1	1	2	2	3	4	4	5
99	99.50	99.55	99.60	99.65	99.70	99.75	99.80	99.85	99.90	99.95	0	1	1	2	2	3	4	4	5

# REDPATH, BROWN & CO., LIMITED.

## CUBES OF NUMBERS FROM 1 TO 249.

No.	Cube.	No.	Cube.	No.	Cube.	No.	Cube.	No.	Cube.
1	1	50	125000	100	1000000	150	3375000	200	8000000
2	8	51	132651	101	1030301	151	3442951	201	8120801
3	27	52	140608	102	1061208	152	3511808	202	8243408
4	64	53	148877	103	1092727	153	3581577	203	8365427
5	125	54	157464	104	1124864	154	3652264	204	8489064
6	216	55	166375	105	1157625	155	3723875	205	8614125
7	343	56	175616	106	1191016	156	3796416	206	8741816
8	512	57	185193	107	1225043	157	3869888	207	8869743
9	729	58	195112	108	1259712	158	3944312	208	8998912
		59	205379	109	1295029	159	4019679	209	9129329
10	1000	60	216000	110	1331000	160	4096000	210	9261000
11	1331	61	226981	111	1367631	161	4173261	211	9393981
12	1728	62	238328	112	1404928	162	4251528	212	9528128
13	2197	63	250047	113	1442897	163	4330747	213	9663527
14	2744	64	262144	114	1481544	164	4410944	214	9800344
15	3375	65	274625	115	1520875	165	4492125	215	9938575
16	4096	66	287496	116	1560896	166	4574296	216	10077696
17	4913	67	300703	117	1601613	167	4657463	217	10218313
18	5832	68	314432	118	1643032	168	4741632	218	10360232
19	6859	69	328569	119	1685169	169	4826809	219	10503469
20	8000	70	343000	120	1728000	170	4913000	220	10648000
21	9261	71	357911	121	1771561	171	6000211	221	10793861
22	10648	72	373248	122	1815848	172	6088448	222	10941048
23	12167	73	389017	123	1860867	173	5177717	223	11089667
24	13824	74	405224	124	1906624	174	6268024	224	11239424
25	15625	75	421875	125	1953125	175	6359375	225	11390625
26	17576	76	438976	126	2000376	176	5451776	226	11543176
27	19683	77	456533	127	2048383	177	6545283	227	11697083
28	21952	78	474552	128	2097152	178	6639762	228	11852352
29	24389	79	493039	129	2146689	179	6735339	229	12008989
30	27000	80	512000	130	2197000	180	5832000	230	12167000
31	29791	81	531441	131	2248091	181	5929741	231	12326391
32	32768	82	551308	132	2299968	182	6028568	232	12487168
33	35937	83	571787	133	2352637	183	6128487	233	12649337
34	39304	84	592704	134	2406104	184	6229504	234	12812904
35	42875	85	614125	135	2460675	185	6331625	235	12977875
36	46656	86	636066	136	2515456	186	6434856	236	13144256
37	50653	87	658503	137	2571353	187	6539203	237	13312063
38	54872	88	681472	138	2628372	188	6644672	238	13481272
39	59319	89	704969	139	2685619	189	6751269	239	13651919
40	64000	90	729000	140	2744000	190	6859000	240	13824000
41	68921	91	753571	141	2803321	191	6967871	241	13997521
42	74088	92	778688	142	2863628	192	7077888	242	14172488
43	79507	93	804367	143	2924207	193	7189067	243	14349807
44	85184	94	830584	144	2985984	194	7301384	244	14528674
45	91125	95	857375	145	3048925	195	7414875	245	14709125
46	97336	96	884736	146	3112136	196	7529536	246	14891286
47	103823	97	912673	147	3176523	197	7645373	247	15075023
48	110592	98	941192	148	3241792	198	7762392	248	15260392
49	117649	99	970299	149	3307949	199	7880599	249	15447349

# REDPATH, BROWN & CO., LIMITED.

## CUBES OF NUMBERS FROM 250 TO 499.

No.	Cube.	No.	Cube.	No.	Cube.	No.	Cube.	No.	Cube.
250	15625000	300	27000000	350	42875000	400	64000000	450	91125000
251	15813251	301	27270901	351	43243551	401	64481201	451	91783551
252	16003008	302	27543608	352	43614208	402	64964808	452	92346408
253	16194277	303	27818127	353	43986977	403	65450827	453	92959677
254	16387064	304	28094464	354	44361864	404	65939264	454	93576064
255	16581375	305	28372625	355	44738875	405	66430125	455	94196375
256	16777216	306	28652616	356	45118016	406	66923416	456	94818816
257	16974593	307	28934443	357	45499293	407	67419143	457	95443893
258	17173512	308	29218112	358	45882712	408	67917312	458	96071912
259	17373979	309	29503629	359	46268279	409	68417929	459	96702579
260	17576000	310	29791000	360	46656000	410	68921000	460	97336000
261	17779581	311	30080231	361	47045881	411	69426531	461	97972181
262	17984728	312	30371328	362	47437928	412	69934528	462	98611128
263	18191447	313	30664297	363	47832147	413	70444997	463	99252847
264	18399744	314	30959144	364	48228544	414	70957944	464	99897344
265	18609625	315	31255875	365	48627125	415	71473375	465	100544625
266	18821096	316	31554496	366	49027896	416	71991296	466	101194696
267	19034163	317	31855013	367	49430863	417	72511713	467	101847563
268	19248832	318	32157432	368	49836032	418	73034632	468	102503232
269	19465109	319	32461759	369	50243409	419	73550059	469	103161709
270	19683000	320	32768000	370	50653000	420	74068800	470	103823000
271	19902511	321	33076161	371	51064811	421	74618461	471	104487111
272	20123648	322	33386248	372	51478448	422	75151448	472	105154048
273	20346417	323	33698367	373	51895117	423	75686967	473	105823817
274	20570824	324	34012224	374	52313624	424	76225024	474	106496424
275	20796875	325	34328125	375	52734375	425	76765625	475	107171875
276	21024576	326	34645976	376	53157376	426	77308776	476	107850176
277	21253933	327	34965733	377	53582633	427	77854483	477	108531333
278	21484952	328	35287552	378	54010152	428	78402752	478	109215352
279	21717639	329	35611289	379	54439939	429	78953589	479	109902239
280	21952000	330	35937000	380	54872000	430	79507000	480	110592000
281	22188041	331	36264691	381	55300341	431	80062991	481	111284641
282	22425768	332	36594388	382	55724268	432	80621568	482	111980168
283	22665187	333	36926037	383	56151837	433	81182737	483	112678587
284	22906304	334	37259704	384	56582104	434	81746504	484	113379904
285	23149125	335	37595375	385	57006625	435	82312875	485	114084125
286	23393656	336	37933056	386	57512456	436	82881856	486	114791256
287	23639903	337	38272753	387	57960603	437	83453453	487	115501303
288	23887872	338	38614472	388	58411072	438	84027672	488	116214272
289	24137569	339	38958219	389	58863869	439	84604519	489	116930169
290	24389000	340	39304000	390	59319000	440	85184000	490	117649000
291	24642171	341	39651821	391	59776471	441	85766121	491	118370771
292	24897088	342	40001688	392	60236288	442	86350688	492	119095488
293	25153757	343	40353707	393	60698457	443	86937857	493	119823157
294	25412184	344	40707584	394	61162984	444	87528384	494	120553784
295	25672375	345	41063625	395	61629375	445	88121125	495	121287375
296	25934336	346	41421736	396	62099136	446	88716536	496	122023936
297	26198073	347	41781923	397	62570773	447	89314623	497	122763473
298	26463592	348	42144192	398	63044792	448	89915392	498	123505992
299	26730899	349	42508549	399	63521199	449	90518849	499	124251499

# REDPATH, BROWN & CO., LIMITED.

## CUBES OF NUMBERS FROM 500 TO 749.

No.	Cube.	No.	Cube.	No.	Cube.	No.	Cube.	No.	Cube.
500	125000000	550	166375000	600	216000000	650	274625000	700	343000000
501	125761501	551	167284151	601	217081801	651	275894551	701	344472101
502	126566008	552	168196808	602	218167208	652	277107808	702	345943408
503	127368507	553	169112377	603	219256227	653	278445077	703	347428227
504	128169404	554	170031464	604	220348864	654	279726364	704	348913664
505	128978705	555	170953875	605	221446125	655	281011375	705	350404625
506	129564216	556	171879616	606	222546016	656	282300416	706	351895816
507	130328343	557	172808693	607	223648543	657	283603393	707	353388243
508	131096512	558	173741112	608	224755712	658	284930812	708	354889412
509	131872220	559	174670879	609	225866529	659	286101179	709	356400829
510	132651000	560	175616000	610	226981000	660	287496000	710	357911000
511	133432831	561	176566481	611	228099131	661	288904781	711	359425431
512	134217728	562	177504328	612	229220928	662	290117528	712	360944128
513	135005697	563	178453547	613	230346397	663	291343247	713	362467097
514	135796744	564	179406144	614	231476544	664	292574244	714	363994344
515	136590875	565	180362125	615	232608375	665	294079625	715	365525875
516	137388096	566	181321496	616	233744896	666	295408296	716	367061096
517	138188413	567	182284263	617	234885113	667	296740963	717	368601813
518	138991832	568	183250432	618	236029032	668	298077632	718	370146232
519	139798359	569	184220009	619	237176659	669	299418309	719	371694959
520	140608000	570	185193000	620	238328000	670	300763000	720	373248000
521	141420761	571	186169411	621	239483061	671	302111711	721	374805361
522	142236648	572	187149248	622	240641848	672	303464448	722	376367048
523	143055667	573	188132517	623	241804367	673	304821217	723	377933067
524	143877824	574	189119234	624	242970624	674	306182024	724	379503424
525	144703125	575	190109375	625	244140625	675	307546875	725	381078125
526	145531576	576	191102976	626	245314376	676	308915776	726	382657176
527	146363183	577	192100033	627	246491883	677	310288733	727	384240683
528	147197952	578	193100552	628	247673152	678	311666752	728	385828352
529	148035889	579	194104539	629	248855189	679	313046839	729	387420489
530	148877000	580	195112000	630	250047000	680	314432000	730	389017000
531	149721291	581	196122941	631	251239591	681	315821241	731	390617891
532	150568768	582	197137368	632	252435068	682	317214568	732	392223168
533	151419437	583	198155287	633	253633017	683	318611987	733	393832837
534	152273304	584	199176704	634	254844014	684	320013604	734	395446904
535	153130375	585	200201625	635	256047875	685	321419125	735	397065375
536	153990666	586	201230066	636	257259466	686	322828566	736	398688266
537	154854163	587	202262003	637	258474853	687	324242703	737	400315653
538	155720872	588	203297472	638	259694072	688	325660672	738	401947272
539	156590819	589	204336469	639	260917119	689	327082769	739	403583419
540	157464000	590	205379000	640	262144000	690	328509000	740	405224000
541	158340421	591	206425071	641	263374721	691	329939371	741	406869021
542	159220088	592	207474688	642	264609288	692	331373888	742	408518488
543	160103007	593	208527857	643	265847707	693	332812567	743	410172707
544	160989184	594	209584584	644	267089984	694	334255684	744	411830784
545	161878625	595	210644875	645	268336125	695	335702875	745	413493625
546	162771336	596	211708736	646	269586136	696	337153536	746	415160936
547	163667323	597	212776173	647	270840023	697	338608873	747	416832723
548	164566692	598	213847192	648	272097792	698	340068392	748	418508992
549	165469149	599	214921799	649	273359449	699	341532099	749	420189749

# REDPATH, BROWN & CO., LIMITED.

## CUBES OF NUMBERS FROM 750 TO 999.

No.	Cube.	No.	Cube.	No.	Cube.	No.	Cube.	No.	Cube.
750	421375000	800	512000000	850	611127000	900	729000000	950	857375000
751	423564751	801	513923401	851	612950551	901	731432701	951	859085351
752	425750008	802	515849608	852	614770208	902	733870308	952	860801408
753	428957777	803	517781037	853	620650477	903	736314327	953	862523177
754	428001064	804	519718464	854	622535504	904	738763264	954	864250664
755	430388875	805	521660125	855	625026375	905	741217625	955	870853875
756	432081216	806	523606616	856	627223016	906	743677416	956	873722816
757	433798008	807	525557043	857	629422793	907	746142643	957	876467493
758	435519612	808	527514112	858	631628712	908	748613312	958	879217912
759	437245479	809	529475129	859	633839779	909	751089429	959	881974079
760	438079000	810	531441000	860	636056000	910	753571000	960	884738000
761	440711081	811	533411731	861	638277381	911	756058931	961	887503681
762	442450728	812	535387328	862	640503928	912	758550528	962	890277128
763	444194947	813	537367797	863	642735647	913	761048497	963	893056637
764	445943744	814	539353144	864	644972644	914	763551944	964	895841344
765	447697125	815	541343375	865	647214025	915	766060875	965	898632125
766	449455096	816	543338496	866	649461896	916	768575296	966	901428096
767	451217663	817	545339513	867	651714363	917	771095213	967	904231063
768	452984832	818	547343432	868	653972032	918	773620632	968	907039232
769	454756609	819	549353269	869	656231809	919	776151559	969	909853209
770	456533000	820	551368000	870	658503000	920	778688000	970	912673000
771	458314011	821	553387601	871	660776311	921	781229061	971	915490661
772	460099648	822	555412248	872	663054848	922	783777448	972	918330048
773	461889017	823	557441767	873	665339817	923	786330467	973	921167317
774	463683424	824	559476224	874	667627024	924	788880224	974	924010424
775	465484375	825	561515625	875	669921575	925	791453125	975	926856975
776	467288576	826	563559976	876	672221376	926	794022776	976	929714176
777	469097433	827	565609233	877	674526133	927	796595933	977	932574833
778	470910952	828	567663552	878	676836152	928	799178752	978	935441352
779	472729139	829	569722789	879	679151439	929	801765089	979	938313739
780	474552000	830	571787000	880	681472000	930	804357000	980	941192000
781	476379541	831	573817691	881	683797841	931	806954191	981	944076141
782	478211768	832	575850368	882	686128968	932	809557568	982	946960168
783	480048887	833	577890937	883	688465387	933	812166237	983	949860987
784	481890304	834	579938704	884	690807104	934	814780604	984	952763904
785	483736625	835	582128275	885	693154125	935	817400375	985	955671625
786	485587056	836	584277056	886	695506456	936	820025856	986	958585256
787	487443403	837	586376253	887	697861103	937	822656953	987	961504803
788	489303872	838	588430472	888	700227072	938	825293672	988	964430272
789	491169000	839	590589710	889	702595369	939	827936019	989	967361609
790	493039000	840	592704000	890	704969000	940	830554000	990	970298000
791	494913671	841	594823371	891	707317971	941	833237621	991	973242271
792	496793088	842	596947688	892	709732288	942	835969888	992	976191488
793	498677257	843	599077107	893	712121057	943	838561507	993	979146657
794	500566184	844	601211584	894	714516984	944	841222384	994	982107774
795	502459975	845	603351125	895	716917375	945	843808625	995	985074875
796	504358736	846	605495736	896	719323136	946	846509056	996	988047936
797	506261573	847	607645423	897	721734273	947	849231823	997	991028673
798	508169592	848	609800192	898	724150792	948	851971392	998	994011992
799	510082369	849	611960049	899	726572899	949	854670349	999	997002369

# REDPATH, BROWN & CO., LIMITED.

## CUBES OF NUMBERS AND FRACTIONAL PARTS.

No.	0	$\frac{1}{8}$	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	No.
0		0019	0156	0527	1250	2441	4219	6899	0
1	1	1424	1953	2600	3375	4291	5359	6592	1
2	8	9596	11391	13396	15625	18088	20797	23764	2
3	27	30518	34328	38443	42875	47635	52734	58186	3
4	64	70189	76706	83740	91125	98932	107172	115857	4
5	125	134611	144703	155287	166375	177979	190109	202779	5
6	216	229783	244141	259084	274625	290775	307547	324951	6
7	343	361705	381078	401131	421875	443322	465484	488373	7
8	512	536377	561510	587428	614125	641619	669922	699045	8
9	729	759799	791453	823975	857375	891666	926859	962967	9
10	1000	103797	107689	111677	115762	119946	124230	128614	10
11	1331	137689	142383	147182	152087	157101	162223	167456	11
12	1728	178256	183827	189511	195312	201231	207267	213423	12
13	2197	226099	232620	239266	246037	252935	259961	267115	13
14	2744	281816	289364	297046	304862	312815	320905	329133	14
15	3375	346008	354668	363451	372367	381410	390588	400005	15
16	4096	419275	429102	439080	449212	459499	469942	480542	16
17	4913	502217	513295	524535	535937	547504	559238	571134	17
18	5832	595435	607839	620415	633162	646084	659180	672451	18
19	6859	699527	713333	727319	741487	755838	770373	785094	19
20	8000	815094	830377	845849	861512	877368	893417	909661	20
21	9261	942736	959570	976604	993837	101127	102891	104675	21
22	10648	108305	110151	112018	113906	115815	117745	119697	22
23	12167	123605	125568	127519	129779	131861	133965	136091	23
24	13824	140411	142805	144822	147061	149324	151609	153918	24
25	15625	158605	160985	163387	165814	168264	170739	173237	25
26	17576	178307	180879	183475	186096	188742	191413	194109	26
27	19683	199576	202348	205146	207970	210818	213692	216593	27
28	21952	222473	225454	228459	231491	234551	237637	240750	28
29	24389	247057	250252	253474	256724	260001	263306	266639	29
30	27000	273389	276890	280252	283726	287229	290760	294321	30

# REDPATH, BROWN & CO., LIMITED.

## CUBES OF NUMBERS AND FRACTIONAL PARTS.

No.	0	$\frac{1}{8}$	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	No.
31	29791	30152'3	30517'6	30885'3	31255'9	31629'4	32006'0	32385'5	31
32	32768	33153'5	33542'0	33933'6	34328'1	34725'7	35126'4	35530'2	32
33	35937	36346'0	36760'0	37176'1	37595'4	38017'8	38443'4	38872'1	33
34	39304	39739'1	40177'4	40618'9	41063'6	41511'6	41962'8	42417'3	34
35	42875	43336'0	43800'3	44267'9	44738'9	45213'1	45690'7	46171'7	35
36	46656	47143'7	47634'8	48129'2	48627'1	49128'4	49633'2	50141'4	36
37	50653	51163'1	51686'7	52208'8	52734'4	53263'5	53796'1	54332'3	37
38	54872	55415'3	55962'1	56512'6	57066'6	57624'3	58185'6	58750'5	38
39	59319	59891'2	60467'1	61046'6	61629'0	62216'3	62807'5	63401'9	39
40	64000	64601'9	65207'5	65816'9	66430'1	67047'1	67667'9	68292'5	40
41	68921	69553'3	70189'5	70829'5	71473'4	72121'2	72772'9	73428'5	41
42	74088	74751'5	75418'9	76090'3	76765'6	77445'0	78128'3	78815'6	42
43	79507	80202'4	80901'8	81605'3	82312'9	83024'5	83740'2	84460'1	43
44	85184	85912'1	86644'3	87380'6	88121'1	88865'8	89614'7	90367'7	44
45	91125	91886'5	92652'2	93422'2	94196'4	94974'8	95757'6	96544'6	45
46	97336	98131'7	98931'6	99736'0	100545	101358	102175	102997	46
47	108823	104654	105489	106323	107172	108020	108873	109730	47
48	110592	111458	112320	113204	114084	114968	115857	116751	48
49	117649	118552	119459	120371	121287	122209	123134	124065	49
50	125000	125940	126884	127834	128788	129740	130710	131678	50
51	132661	133629	134611	135599	136591	137588	138590	139596	51
52	140608	141624	142646	143672	144703	145730	146780	147826	52
53	148877	149933	150994	152060	153130	154206	155287	156373	53
54	157464	158560	159661	160767	161879	162995	164117	165243	54
55	166376	167512	168654	169801	170954	172112	173274	174443	55
56	175616	176795	177979	179168	180362	181562	182767	183977	56
57	185198	186414	187640	188872	190109	191352	192600	193853	57
58	195112	196376	197646	198921	200202	201488	202779	204076	58
59	205379	206687	208001	209320	210645	211975	213311	214653	59
60	216000	217853	218711	220075	221445	222821	224202	225588	60

# REDPATH, BROWN & CO., LIMITED.

## CIRCUMFERENCES OF CIRCLES ADVANCING BY EIGHTHS.

Diam.	0	$\frac{1}{8}$	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	Diam.
0									0
1	8'142	8'327	8'514	9'101	9'287	9'473	9'659	9'845	1
2	6'283	6'476	6'669	6'861	7'054	7'246	7'438	7'630	2
3	9'425	9'817	10'210	10'603	10'996	11'388	11'781	12'174	3
4	12'566	12'959	13'352	13'744	14'137	14'530	14'923	15'316	4
5	16'708	16'101	16'493	16'886	17'279	17'671	18'064	18'457	5
6	18'850	19'242	19'635	20'028	20'420	20'813	21'206	21'598	6
7	21'991	22'384	22'777	23'169	23'562	23'955	24'347	24'740	7
8	25'133	25'525	25'918	26'311	26'704	27'096	27'489	27'882	8
9	28'274	28'667	29'060	29'452	29'845	30'238	30'631	31'023	9
10	31'416	31'809	32'201	32'594	32'987	33'379	33'772	34'165	10
11	34'558	34'950	35'343	35'736	36'128	36'521	36'914	37'306	11
12	37'699	38'092	38'485	38'877	39'270	39'663	40'055	40'448	12
13	40'841	41'233	41'626	42'019	42'412	42'804	43'197	43'590	13
14	43'982	44'375	44'768	45'160	45'553	45'946	46'338	46'731	14
15	47'124	47'517	47'909	48'302	48'695	49'087	49'480	49'873	15
16	50'265	50'658	51'051	51'444	51'836	52'229	52'622	53'014	16
17	53'407	53'800	54'192	54'585	54'978	55'371	55'763	56'156	17
18	56'549	56'941	57'334	57'727	58'119	58'512	58'905	59'298	18
19	59'690	60'083	60'476	60'868	61'261	61'654	62'046	62'439	19
20	62'832	63'225	63'617	64'010	64'403	64'795	65'188	65'581	20
21	65'973	66'366	66'759	67'152	67'544	67'937	68'330	68'722	21
22	69'115	69'508	69'900	70'293	70'686	71'079	71'471	71'864	22
23	72'257	72'649	73'042	73'435	73'827	74'220	74'613	75'006	23
24	75'398	75'791	76'184	76'576	76'969	77'362	77'754	78'147	24
25	78'540	78'933	79'326	79'718	80'111	80'503	80'896	81'289	25
26	81'681	82'074	82'467	82'860	83'252	83'645	84'038	84'430	26
27	84'823	85'216	85'608	86'001	86'394	86'786	87'179	87'572	27
28	87'965	88'357	88'750	89'143	89'535	89'928	90'321	90'713	28
29	91'106	91'499	91'892	92'284	92'677	93'070	93'462	93'855	29
30	94'248	94'640	95'033	95'426	95'819	96'211	96'604	96'997	30
31	97'389	97'782	98'175	98'567	98'960	99'353	99'746	100'14	31
32	100'53	100'92	101'32	101'71	102'10	102'49	102'89	103'28	32
33	103'67	104'07	104'46	104'85	105'24	105'64	106'03	106'42	33
34	106'81	107'21	107'60	107'99	108'38	108'78	109'17	109'56	34
35	109'96	110'35	110'74	111'13	111'53	111'92	112'31	112'70	35
36	113'10	113'49	113'88	114'28	114'67	115'06	115'45	115'85	36
37	116'24	116'63	117'02	117'42	117'81	118'20	118'60	118'99	37
38	119'38	119'77	120'17	120'56	120'95	121'34	121'74	122'13	38
39	122'52	122'91	123'31	123'70	124'09	124'49	124'88	125'27	39
40	125'66	126'06	126'45	126'84	127'23	127'63	128'02	128'41	40
41	128'81	129'20	129'59	129'98	130'38	130'77	131'16	131'55	41
42	131'95	132'34	132'73	133'12	133'52	133'91	134'30	134'70	42
43	135'09	135'48	135'87	136'27	136'66	137'05	137'44	137'84	43
44	138'23	138'62	139'02	139'41	139'80	140'19	140'59	140'98	44
45	141'37	141'76	142'16	142'55	142'94	143'34	143'73	144'12	45
46	144'51	144'91	145'30	145'69	146'08	146'48	146'87	147'26	46
47	147'65	148'05	148'44	148'83	149'23	149'62	150'01	150'40	47
48	150'80	151'19	151'58	151'97	152'37	152'76	153'15	153'55	48
49	153'94	154'33	154'72	155'12	155'51	155'90	156'29	156'69	49
50	157'08	157'47	157'86	158'26	158'65	159'04	159'44	159'83	50

# REDPATH, BROWN & CO., LIMITED.

## CIRCUMFERENCES OF CIRCLES ADVANCING BY EIGHTHS.

Diam.	0	$\frac{1}{8}$	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	Diam.
51	160.22	160.61	161.01	161.40	161.79	162.18	162.58	162.97	51
52	163.36	163.76	164.15	164.54	164.93	165.33	165.72	166.11	52
53	166.50	166.90	167.29	167.68	168.08	168.47	168.86	169.25	53
54	169.66	170.04	170.43	170.82	171.22	171.61	172.00	172.39	54
55	172.79	173.18	173.57	173.97	174.36	174.75	175.14	175.54	55
56	175.93	176.32	176.71	177.11	177.50	177.89	178.29	178.68	56
57	179.07	179.46	179.86	180.25	180.64	181.03	181.43	181.82	57
58	182.21	182.61	183.00	183.39	183.78	184.18	184.57	184.96	58
59	185.35	185.75	186.14	186.53	186.92	187.32	187.71	188.10	59
60	188.60	188.99	189.38	189.77	190.17	190.56	190.95	191.34	60
61	191.64	192.03	192.42	192.82	193.21	193.60	193.99	194.39	61
62	194.78	195.17	195.56	195.96	196.35	196.74	197.13	197.53	62
63	197.92	198.31	198.71	199.10	199.49	199.88	200.28	200.67	63
64	201.06	201.45	201.85	202.24	202.63	203.03	203.42	203.81	64
65	204.20	204.60	204.99	205.38	205.77	206.17	206.56	206.96	65
66	207.35	207.74	208.13	208.52	208.92	209.31	209.70	210.09	66
67	210.49	210.88	211.27	211.66	212.06	212.45	212.84	213.24	67
68	213.63	214.02	214.41	214.81	215.20	215.59	215.98	216.38	68
69	216.77	217.16	217.56	217.95	218.34	218.73	219.13	219.52	69
70	219.91	220.30	220.70	221.09	221.48	221.87	222.27	222.66	70
71	223.05	223.45	223.84	224.23	224.62	225.02	225.41	225.80	71
72	226.19	226.59	226.98	227.37	227.77	228.16	228.55	228.94	72
73	229.34	229.73	230.12	230.51	230.91	231.30	231.69	232.09	73
74	232.48	232.87	233.26	233.66	234.05	234.44	234.83	235.23	74
75	235.62	236.01	236.40	236.80	237.19	237.58	237.98	238.37	75
76	238.76	239.15	239.55	239.94	240.33	240.72	241.12	241.51	76
77	241.90	242.30	242.69	243.08	243.47	243.87	244.26	244.65	77
78	245.04	245.44	245.83	246.22	246.62	247.01	247.40	247.79	78
79	248.19	248.58	248.97	249.36	249.76	250.15	250.54	250.93	79
80	251.83	252.22	252.61	253.01	253.40	253.79	254.19	254.58	80
81	254.47	254.86	255.25	255.65	256.04	256.43	256.83	257.22	81
82	257.61	258.00	258.40	258.79	259.18	259.57	259.97	260.36	82
83	260.76	261.15	261.54	261.93	262.32	262.72	263.11	263.50	83
84	263.89	264.29	264.68	265.07	265.46	265.86	266.25	266.64	84
85	267.04	267.43	267.82	268.21	268.61	269.00	269.39	269.78	85
86	270.18	270.57	270.96	271.36	271.75	272.14	272.53	272.93	86
87	273.32	273.71	274.10	274.50	274.89	275.28	275.67	276.07	87
88	276.46	276.85	277.25	277.64	278.03	278.42	278.82	279.21	88
89	279.60	279.99	280.39	280.78	281.17	281.57	281.96	282.36	89
90	282.74	283.14	283.53	283.93	284.32	284.71	285.10	285.49	90
91	285.88	286.28	286.67	287.06	287.46	287.85	288.24	288.63	91
92	289.03	289.42	289.81	290.20	290.60	290.99	291.38	291.78	92
93	292.17	292.56	292.96	293.35	293.74	294.13	294.52	294.92	93
94	296.31	296.70	297.10	297.49	297.88	298.27	298.67	299.06	94
95	298.45	298.84	299.24	299.63	300.02	300.41	300.81	301.20	95
96	301.59	301.99	302.38	302.77	303.16	303.56	303.95	304.34	96
97	304.73	305.13	305.52	305.91	306.31	306.70	307.09	307.48	97
98	307.88	308.27	308.66	309.05	309.45	309.84	310.23	310.62	98
99	311.02	311.41	311.80	312.20	312.59	312.98	313.37	313.77	99
100	314.16	314.55	314.94	315.34	315.73	316.12	316.52	316.91	100

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## AREAS OF CIRCLES ADVANCING BY EIGHTHS.

Diam.	0	$\frac{1}{8}$	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	Diam.
0		0123	0491	1104	1963	3068	4418	6013	0
1	7854	9940	1227	1485	1767	2074	2405	2761	1
2	3142	8547	8476	4430	4909	5412	5940	6492	2
3	7009	7070	8296	8946	9621	10321	11045	11793	3
4	12566	13364	14186	15031	15904	16800	17721	18665	4
5	19635	20629	21643	22691	23758	24850	25967	27100	5
6	28274	29465	30680	31919	33183	34472	35785	37122	6
7	38485	39871	41282	42718	44179	45664	47173	48707	7
8	50265	51849	53456	55088	56745	58426	60132	61863	8
9	63617	65397	67201	69029	70882	72760	74662	76589	9
10	78540	80516	82516	84541	86590	88664	90763	92886	10
11	95033	97205	99402	10162	10387	10614	10843	11075	11
12	11310	11547	11786	12028	12272	12519	12768	13019	12
13	13273	13530	13789	14050	14314	14580	14848	15118	13
14	15394	15670	15948	16228	16510	16795	17082	17371	14
15	17671	17967	18265	18566	18869	19175	19483	19793	15
16	20108	20422	20739	21060	21382	21708	22035	22365	16
17	22698	23033	23371	23710	24053	24398	24745	25095	17
18	25447	25802	26159	26518	26880	27245	27612	27981	18
19	28353	28727	29104	29483	29865	30249	30635	31024	19
20	31416	31810	32206	32605	33006	33410	33816	34225	20
21	34636	35060	35486	35914	36343	36774	37206	37640	21
22	38013	38446	38882	39320	39761	40204	40649	41097	22
23	41648	42090	42536	42983	43434	43886	44341	44799	23
24	46239	46711	47186	47664	48144	48626	49111	49598	24
25	49037	49579	50124	50671	51221	51772	52325	52880	25
26	53093	53605	54119	54635	55155	55676	56200	56727	26
27	57256	57787	58321	58857	59396	59937	60481	61027	27
28	61575	62126	62680	63236	63794	64355	64918	65484	28
29	66052	66623	67196	67771	68349	68929	69513	70098	29
30	70686	71270	71859	72454	73052	73652	74255	74860	30
31	75477	76079	76687	77294	77911	78531	79153	79778	31
32	80425	81054	81688	82328	82974	83626	84283	84943	32
33	85530	86179	86833	87485	88144	88809	89482	90160	33
34	90792	91461	92132	92806	93482	94161	94842	95525	34
35	96311	96900	97501	98104	98710	99318	99930	100545	35
36	10179	10250	10321	10392	10463	10535	10607	10680	36
37	10752	10825	10898	10971	11045	11118	11192	11267	37
38	11341	11416	11491	11566	11642	11717	11793	11869	38
39	11946	12023	12100	12177	12254	12332	12410	12488	39
40	12566	12645	12724	12803	12882	12962	13042	13122	40
41	13203	13283	13364	13445	13527	13608	13690	13772	41
42	13854	13937	14020	14103	14186	14270	14354	14438	42
43	14522	14607	14691	14776	14862	14947	15033	15119	43
44	15205	15292	15379	15466	15553	15640	15728	15816	44
45	15904	15993	16082	16170	16260	16349	16439	16529	45
46	16619	16709	16800	16891	16982	17074	17165	17257	46
47	17349	17442	17535	17627	17721	17814	17908	18001	47
48	18096	18190	18285	18379	18475	18570	18665	18761	48
49	18857	18954	19050	19147	19244	19342	19439	19537	49
50	19635	19733	19832	19931	20030	20129	20228	20328	50

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## AREAS OF CIRCLES ADVANCING BY EIGHTHS.

Diam.	0	$\frac{1}{8}$	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	Diam.
51	2042.8	2052.8	2062.9	2073.0	2083.1	2093.2	2103.3	2113.5	51
52	2123.7	2133.9	2144.2	2154.5	2164.8	2175.1	2185.4	2195.8	52
53	2204.6	2214.6	2224.7	2234.7	2244.8	2254.8	2264.9	2274.9	53
54	2285.2	2295.2	2305.3	2315.3	2325.4	2335.4	2345.5	2355.5	54
55	2375.8	2385.6	2395.7	2405.8	2415.8	2425.9	2435.9	2445.9	55
56	2465.0	2474.0	2485.0	2496.1	2507.2	2518.3	2529.4	2540.6	56
57	2551.8	2563.0	2574.2	2585.4	2596.7	2608.0	2619.4	2630.7	57
58	2642.1	2653.5	2664.9	2676.4	2687.8	2699.3	2710.9	2722.4	58
59	2734.0	2745.6	2757.2	2768.8	2780.5	2792.2	2803.9	2815.7	59
60	2827.4	2839.2	2851.0	2862.9	2874.8	2886.6	2898.6	2910.5	60
61	2922.5	2934.5	2946.5	2958.5	2970.6	2982.7	2994.8	3006.9	61
62	3019.1	3031.3	3043.5	3055.7	3068.0	3080.3	3092.6	3104.9	62
63	3117.2	3129.6	3142.0	3154.5	3166.9	3179.4	3191.9	3204.4	63
64	3217.0	3229.6	3242.2	3254.8	3267.5	3280.1	3292.8	3305.6	64
65	3318.3	3331.1	3343.9	3356.7	3369.6	3382.4	3395.3	3408.2	65
66	3421.2	3434.2	3447.2	3460.2	3473.2	3486.3	3499.4	3512.5	66
67	3525.7	3538.8	3552.0	3565.2	3578.5	3591.7	3605.0	3618.3	67
68	3631.7	3645.0	3658.4	3671.8	3685.3	3698.7	3712.2	3725.7	68
69	3739.3	3752.8	3766.4	3780.0	3793.7	3807.3	3821.0	3834.7	69
70	3848.6	3862.2	3876.0	3889.8	3903.6	3917.5	3931.4	3945.3	70
71	3959.2	3973.1	3987.1	4001.1	4015.2	4029.2	4043.3	4057.4	71
72	4071.5	4085.7	4099.8	4114.0	4128.2	4142.5	4156.8	4171.1	72
73	4185.4	4199.7	4214.1	4228.5	4242.9	4257.4	4271.8	4286.3	73
74	4300.8	4315.4	4329.9	4344.5	4359.2	4373.8	4388.5	4403.2	74
75	4417.9	4432.6	4447.4	4462.2	4477.0	4491.8	4506.7	4521.5	75
76	4536.5	4551.4	4566.4	4581.3	4596.3	4611.4	4626.4	4641.5	76
77	4656.6	4671.8	4686.9	4702.1	4717.3	4732.5	4747.8	4763.1	77
78	4778.4	4793.7	4809.0	4824.4	4839.8	4855.2	4870.7	4886.2	78
79	4901.7	4917.2	4932.7	4948.3	4963.9	4979.5	4995.2	5010.9	79
80	5026.5	5042.3	5058.0	5073.8	5089.6	5105.4	5121.2	5137.1	80
81	5153.0	5168.9	5184.9	5200.8	5216.8	5232.8	5248.9	5264.9	81
82	5281.0	5297.1	5313.3	5329.4	5345.6	5361.8	5378.1	5394.3	82
83	5410.6	5426.9	5443.3	5459.6	5476.0	5492.4	5508.8	5525.3	83
84	5541.8	5558.3	5574.8	5591.4	5607.9	5624.5	5641.2	5657.8	84
85	5674.5	5691.2	5707.9	5724.7	5741.5	5758.3	5775.1	5791.9	85
86	5808.8	5825.7	5842.6	5859.6	5876.5	5893.5	5910.6	5927.6	86
87	5944.7	5961.8	5978.9	5996.0	6013.2	6030.4	6047.6	6064.9	87
88	6082.1	6099.4	6116.7	6134.1	6151.4	6168.8	6186.2	6203.7	88
89	6221.1	6238.6	6256.1	6273.7	6291.2	6308.8	6326.4	6344.1	89
90	6361.7	6379.4	6397.1	6414.9	6432.6	6450.4	6468.2	6486.0	90
91	6503.9	6521.8	6539.7	6557.6	6575.5	6593.5	6611.5	6629.6	91
92	6647.0	6665.0	6683.0	6701.1	6720.1	6738.2	6756.4	6774.7	92
93	6792.9	6811.2	6829.5	6847.8	6866.1	6884.5	6902.9	6921.3	93
94	6939.8	6958.2	6976.7	6995.3	7013.8	7032.4	7051.0	7069.6	94
95	7088.2	7106.9	7125.6	7144.3	7163.0	7181.8	7200.6	7219.4	95
96	7238.2	7257.1	7276.0	7294.0	7313.8	7332.8	7351.8	7370.8	96
97	7389.3	7408.9	7428.0	7447.1	7466.2	7485.3	7504.5	7523.7	97
98	7543.0	7562.2	7581.5	7600.8	7620.1	7639.5	7658.9	7678.3	98
99	7697.7	7717.1	7736.6	7756.1	7775.6	7795.2	7814.8	7834.4	99
100	7854.0	7873.6	7893.3	7913.0	7932.7	7952.5	7972.2	7992.0	100

## USE OF THE MATHEMATICAL TABLES

(pages 332 to 353).

### LOGARITHMS AND ANTILOGARITHMS

(pages 332 to 335).

The logarithm or "log" of a number consists of an integer and a decimal.

The Integral Part is referred to as the Index; and is determined by the following rules:—

Rule (a) If the number whose log is required contain one or more integral figures the index is always less by one than the number of integral figures in the number and is always positive.

Rule (b) If the number is wholly a decimal the index is numerically greater by one than the number of ciphers after the decimal point and is always negative.

The Decimal Part of a log is called the Mantissa and is found from the Tables, pages 332 and 333.

#### TO FIND THE LOG OF A GIVEN NUMBER OF 4 DIGITS.

Ascertain index by rules (a) or (b) above.

Rule (c) To find mantissa, find the first two digits of the given number in left hand column. Pass along horizontal line and read number in vertical line headed by third digit. Add the number in the same horizontal line in the "Mean Differences" column headed by the fourth digit. The result is the required mantissa which, with the index and decimal point prefixed, is the required log.

*Examples—*

(1). Required log 4875.0.

There are 4 figures before decimal point  $\therefore$  index = 3

From log tables, - 487 = .6875

Difference for 5 = 4

log 4875.0 = 3.6875

2). Required log .04875.

There is 1 cipher after decimal point  $\therefore$  index =  $\bar{2}$  (i.e. - 2).

From log tables, - 487 = .6875

Difference for 5 = 4

log .04875 =  $\bar{2}$ .6875

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## LOGS AND ANTILOGS—(continued).

**Negative Index**—Note that the mantissa of a log is always positive, hence the log of a decimal is the algebraic sum of a positive mantissa and a negative index. Thus:—

$$\bar{2}.6879 = .6879 - 2.$$

## ANTILOGARITHMS

(pages 334, 335).

Having obtained the log of any expression, to find the number corresponding to this log use the antilog tables in a similar manner to that described above.

**NOTE.**—In referring to the antilog tables the index of the given log has not to be considered, only the mantissa being used. Having obtained the sequence of figures corresponding to the latter, the index is used to fix the position of the decimal point by the converse of Rules (a) and (b).

### Examples—

- (3) Find number corresponding to the log 5.6879.

From antilog tables number = 4875.

Number of figures = (index + 1) = 5 + 1 = 6.

∴ Required number = 487,500.

- (4) Find number corresponding to the log  $\bar{3}.5503$ .

From antilog tables number = 3551.

Number of ciphers = (index - 1) = 3 - 1 = 2.

∴ Required number = .003551.

## TO PERFORM MULTIPLICATION BY USE OF LOGS.

**Rule (d)** Add the logs of the factors.

**Example (5)**—Required  $3.551 \times .04875$ .

$$\log 3.551 = .5503.$$

$$\log .04875 = \bar{2}.6879.$$

$$\text{Sum} = \bar{1}.2382. \quad (\text{Note manipulation of indices}).$$

$$\text{Product} = \text{antilog of sum.}$$

$$= .1732.$$

## TO PERFORM DIVISION BY USE OF LOGS.

**Rule (e)** Subtract the log of the divisor from the log of the dividend. The remainder is the log of the quotient.

**Example (6)**—Required  $.3551 \div 48.75$

$$\log .3551 = \bar{1}.5503$$

$$\log 48.75 = 1.6879$$

(Note manipulation of negative indices.)

$$\text{Remainder} = \bar{3}.8624$$

$$\text{Quotient} = \text{antilog of remainder.}$$

$$= .007285.$$

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## LOGS AND ANTILOGS—(continued).

### TO FIND ANY POWER OF A NUMBER BY USE OF LOGS.

Rule (f) Multiply the log of the number by the exponent of the power to which it is to be raised. This gives the log of the required power.

*Example (7)*—Required  $(3.551)^4$   
 $\log 3.551 = .5503$   
 $\text{Product} = 2.2012$   
 Power = antilog of product  
 $= 159.0$

### TO FIND ANY ROOT OF A NUMBER BY USE OF LOGS.

Rule (g) Divide the log of the number by the exponent of the root which is to be extracted. This gives the log of the required root.

*Example (8)*—Required  $\sqrt[3]{4.875}$ .  
 $\log 4.875 = .6879$   
 $\text{Quotient} = \frac{.6879}{3} = .2293$   
 Root = antilog of quotient  
 $= 1.695$

*Example (9)*—Required  $\sqrt[5]{.004041}$ .  
 $\log .004041 = 3.6065$   
 $= 6065 - 3$ . (Note manipulation of indices).  
 $= 2.6065 - 5$ .  
 $\frac{2.6065 - 5}{5} = .5213 - 1$ .  
 Quotient =  $\bar{1}.5213$ .  
 Root = antilog of quotient.  
 $= .3321$ .

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## MATHEMATICAL TABLES—(continued).

### TRIGONOMETRICAL TABLES

• (pages 336 to 347).

The tables of sines, cosines, &c., give the required function for any angle less than  $90^\circ$ .

The method of using the tables is similar to that described for logs.

In the *sine* and *cosine* tables the decimal point is not shown, as all values are less than unity.

Care must be taken that in using the *cosine*, *cosecant*, and *cotangent* tables for an angle containing an odd number of minutes, the mean difference be subtracted, not added as in the other tables.

In the *tangent*, *cotangent*, *cosecant*, and *secant* tables, the decimal point is shown in the first column only, except in the cases where the variation of the function is so rapid that an approximate value only can be given. The integer (if any) is also given in the first column. The latter is to be prefixed to all values in the corresponding horizontal line with the following exception:—

Rule (h) Where values in any of the horizontal lines are printed in *italics* the integer to be prefixed is—

(a) When differences have to be added, greater by one than the number in the left hand column corresponding.

(b) When the differences have to be subtracted, less by one than the number in the left hand column corresponding.

*Example* (10)—Required  $\tan 63^\circ 25'$   
From tables  $\tan 63^\circ 24' = 1.9970$   
Difference for  $1' = \quad 15$   
 $\tan 63^\circ 25' = 1.9985$

*Example* (11)—Required  $\tan 63^\circ 32'$ .  
In the tables opposite  $63^\circ$  and under  $30'$  read *0057*.  
Corresponding integer in first column is 1.

$\therefore \tan 63^\circ 30' = 2.0057$   
Difference for  $2' = \quad 29$   
 $\tan 63^\circ 32' = 2.0086$

Note that the addition or subtraction of the difference for odd minutes occasionally causes the alteration of the corresponding integer as in example (12).

*Example* (12)—Required  $\tan 63^\circ 27'$   
From tables,  $\tan 63^\circ 24' = 1.9970$   
Difference for  $3' = \quad 44$   
 $\therefore \tan 63^\circ 27' = 2.0014$

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## MATHEMATICAL TABLES—(concluded).

### SQUARES AND SQUARE ROOTS

(pages 348 to 353).

The Tables of Squares afford a rapid means of finding approximately the square of any number from 1 to 9999.

The tables are used as previously described for logs. The position of the decimal point is fixed by a rough mental calculation.

*Example (13)*—Required  $(98.3)^2$ .

Opposite 9.8 and under 3 read 96.63.

Now  $98.3 = 10 \times 9.83$ .  $\therefore (98.3)^2 = 10^2(9.83)^2 = 100 \times 96.63$   
 $= 9663.$

**Tables of Square Roots.** It will be noticed that two tables of square roots are given.

**Rule (i)** TABLE 100 to 999.9 (pages 350, 351) will be used:—

- (a) for the square root of a number greater than unity if the number of digits before the decimal point is odd;
- (b) for the square root of a number less than unity if the number of ciphers after the decimal point is odd.

**Rule (k)** TABLE 1000 to 9999 (pages 352, 353) will be used:—

- (c) for the square root of a number greater than unity if the number of digits before the decimal point is even;
- (d) for the square root of a number less than unity if the number of ciphers after the decimal point is zero or even.

A rough mental calculation will enable the position of the decimal point in the square to be fixed.

*Examples—*

(14) Required  $\sqrt{983.1}$

There are 3 digits before point,  $\therefore$  Table 100 to 999.9 is used.  
Square Root = 31.35.

(15) Required  $\sqrt{.09831}$

There is one cipher after point,  $\therefore$  same Table is used.  
Square Root = .3135.

(16) Required  $\sqrt{98.31}$

There are 2 digits before point,  $\therefore$  Table 1000 to 9999 is used.  
Square Root = 9.915.

(17) Required  $\sqrt{.9831}$

There are no ciphers after point,  $\therefore$  same Table is used.  
Square Root = .9915.

# REDPATH, BROWN & CO., LIMITED.

## DECIMALS OF AN INCH FOR EACH 1/64th.

1/32nds.	1/64ths.	Decimal.	Fraction.	1/32nds.	1/64ths.	Decimal.	Fraction.
	1	.015625			33	.515625	
1	2	.03125		17	34	.53125	
	3	.046875			35	.546875	
2	4	.0625	1—16	18	36	.5625	9—16
	5	.078125			37	.578125	
3	6	.09375		19	38	.59375	
	7	.109375			39	.609375	
4	8	.125	1—8	20	40	.625	5—8
	9	.140625			41	.640625	
5	10	.15625		21	42	.65625	
	11	.171875			43	.671875	
6	12	.1875	3—16	22	44	.6875	11—16
	13	.203125			45	.703125	
7	14	.21875		23	46	.71875	
	15	.234375			47	.734375	
8	16	.25	1—4	24	48	.75	3—4
	17	.265625			49	.765625	
9	18	.28125		25	50	.78125	
	19	.296875			51	.796875	
10	20	.3125	5—16	26	52	.8125	13—16
	21	.328125			53	.828125	
11	22	.34375		27	54	.84375	
	23	.359375			55	.859375	
12	24	.375	3—8	28	56	.875	7—8
	25	.390625			57	.890625	
13	26	.40625		29	58	.90625	
	27	.421875			59	.921875	
14	28	.4375	7—16	30	60	.9375	15—16
	29	.453125			61	.953125	
15	30	.46875		31	62	.96875	
	31	.484375			63	.984375	
16	32	.5	1—2	32	64	1	1

# REDPATH, BROWN & CO., LIMITED.

## DECIMALS OF A FOOT.

For each 1/64th of an inch.

Inch.	0'	1"	2"	3"	4"	5"	6"	7"	8"	9"	10'	11"
0		.0833	.1667	.2500	.3333	.4167	.5000	.5833	.6667	.7500	.8333	.9167
1/16	.0013	.0846	.1680	.2513	.3346	.4180	.5013	.5846	.6680	.7513	.8346	.9180
1/8	.0026	.0859	.1693	.2526	.3359	.4193	.5026	.5859	.6693	.7526	.8359	.9193
3/16	.0039	.0872	.1706	.2539	.3372	.4206	.5039	.5872	.6706	.7539	.8372	.9206
1/4	.0052	.0885	.1719	.2552	.3385	.4219	.5052	.5885	.6719	.7552	.8385	.9219
5/16	.0065	.0898	.1732	.2565	.3398	.4232	.5065	.5898	.6732	.7565	.8398	.9232
3/8	.0078	.0911	.1745	.2578	.3411	.4245	.5078	.5911	.6745	.7578	.8411	.9245
7/16	.0091	.0924	.1758	.2591	.3424	.4258	.5091	.5924	.6758	.7591	.8424	.9258
1/2	.0104	.0937	.1771	.2604	.3437	.4271	.5104	.5937	.6771	.7604	.8437	.9271
5/8	.0117	.0951	.1784	.2617	.3451	.4284	.5117	.5951	.6784	.7617	.8451	.9284
3/4	.0130	.0964	.1797	.2630	.3464	.4297	.5130	.5964	.6797	.7630	.8464	.9297
7/8	.0143	.0977	.1810	.2643	.3477	.4310	.5143	.5977	.6810	.7643	.8477	.9310
15/16	.0156	.0990	.1823	.2656	.3490	.4323	.5156	.5990	.6823	.7656	.8490	.9323
1	.0169	.1003	.1836	.2669	.3503	.4336	.5169	.6003	.6836	.7669	.8503	.9336
1 1/16	.0182	.1016	.1849	.2682	.3516	.4349	.5182	.6016	.6849	.7682	.8516	.9349
1 1/8	.0195	.1029	.1862	.2695	.3529	.4362	.5195	.6029	.6862	.7695	.8529	.9362
1 3/16	.0208	.1042	.1875	.2708	.3542	.4375	.5208	.6042	.6875	.7708	.8542	.9375
1 1/4	.0221	.1055	.1888	.2721	.3555	.4388	.5221	.6055	.6888	.7721	.8555	.9388
1 5/16	.0234	.1068	.1901	.2734	.3568	.4401	.5234	.6068	.6901	.7734	.8568	.9401
1 3/8	.0247	.1081	.1914	.2747	.3581	.4414	.5247	.6081	.6914	.7747	.8581	.9414
1 7/16	.0260	.1094	.1927	.2760	.3594	.4427	.5260	.6094	.6927	.7760	.8594	.9427
1 1/2	.0273	.1107	.1940	.2773	.3607	.4440	.5273	.6107	.6940	.7773	.8607	.9440
1 5/8	.0286	.1120	.1953	.2786	.3620	.4453	.5286	.6120	.6953	.7786	.8620	.9453
1 3/4	.0299	.1133	.1966	.2799	.3633	.4466	.5299	.6133	.6966	.7799	.8633	.9466
1 7/8	.0312	.1146	.1979	.2812	.3646	.4479	.5312	.6146	.6979	.7812	.8646	.9479
2	.0326	.1159	.1992	.2826	.3659	.4492	.5326	.6159	.6992	.7826	.8659	.9492
2 1/16	.0339	.1172	.2005	.2839	.3672	.4505	.5339	.6172	.7005	.7839	.8672	.9505
2 1/8	.0352	.1185	.2018	.2852	.3685	.4518	.5352	.6185	.7018	.7852	.8685	.9518
2 3/16	.0365	.1198	.2031	.2865	.3698	.4531	.5365	.6198	.7031	.7865	.8698	.9531
2 1/4	.0378	.1211	.2044	.2878	.3711	.4544	.5378	.6211	.7044	.7878	.8711	.9544
2 5/16	.0391	.1224	.2057	.2891	.3724	.4557	.5391	.6224	.7057	.7891	.8724	.9557
2 3/8	.0404	.1237	.2070	.2904	.3737	.4570	.5404	.6237	.7070	.7904	.8737	.9570
2 7/16	.0417	.1250	.2083	.2917	.3750	.4583	.5417	.6250	.7083	.7917	.8750	.9583

# REDPATH, BROWN & CO., LIMITED.

## DECIMALS OF A FOOT.

For each 1/64th of an inch.

Inch.	0"	1"	2"	3"	4"	5"	6"	7"	8"	9"	10"	11"
11	0490	1268	2096	2980	3763	4596	5430	6263	7096	7930	8763	9596
11	0443	1276	2109	2943	3776	4609	5443	6276	7109	7943	8776	9609
11	0456	1289	2122	2956	3789	4622	5456	6289	7122	7956	8789	9622
11	0469	1302	2135	2969	3802	4635	5469	6302	7135	7969	8802	9635
11	0482	1315	2148	2982	3815	4648	5482	6315	7148	7982	8815	9648
11	0495	1328	2161	2995	3828	4661	5495	6328	7161	7995	8828	9661
11	0508	1341	2174	3008	3841	4674	5508	6341	7174	8008	8841	9674
11	0521	1354	2188	3021	3854	4688	5521	6354	7188	8021	8854	9688
11	0534	1367	2201	3034	3867	4701	5534	6367	7201	8034	8867	9701
11	0547	1380	2214	3047	3880	4714	5547	6380	7214	8047	8880	9714
11	0560	1393	2227	3060	3893	4727	5560	6393	7227	8060	8893	9727
11	0573	1406	2240	3073	3906	4740	5573	6406	7240	8073	8906	9740
11	0586	1419	2253	3086	3919	4753	5586	6419	7253	8086	8919	9753
11	0599	1432	2266	3099	3932	4766	5599	6432	7266	8099	8932	9766
11	0612	1445	2279	3112	3945	4779	5612	6445	7279	8112	8945	9779
11	0625	1458	2292	3125	3958	4792	5625	6458	7292	8125	8958	9792
11	0638	1471	2305	3138	3971	4805	5638	6471	7305	8138	8971	9805
11	0651	1484	2318	3151	3984	4818	5651	6484	7318	8151	8984	9818
11	0664	1497	2331	3164	3997	4831	5664	6497	7331	8164	8997	9831
11	0677	1510	2344	3177	4010	4844	5677	6510	7344	8177	9010	9844
11	0690	1523	2357	3190	4023	4857	5690	6523	7357	8190	9023	9857
11	0703	1536	2370	3203	4036	4870	5703	6536	7370	8203	9036	9870
11	0716	1549	2383	3216	4049	4883	5716	6549	7383	8216	9049	9883
11	0729	1562	2396	3229	4062	4896	5729	6562	7396	8229	9062	9896
11	0742	1575	2409	3242	4075	4909	5742	6575	7409	8242	9075	9909
11	0755	1588	2422	3255	4089	4922	5755	6589	7422	8255	9089	9922
11	0768	1602	2435	3268	4102	4935	5768	6602	7435	8268	9102	9935
11	0781	1615	2448	3281	4115	4948	5781	6615	7448	8281	9115	9948
11	0794	1628	2461	3294	4128	4961	5794	6628	7461	8294	9128	9961
11	0807	1641	2474	3307	4141	4974	5807	6641	7474	8307	9141	9974
11	0820	1654	2487	3320	4154	4987	5820	6654	7487	8320	9154	9987
1												1-0000

# REDPATH, BROWN & CO., LIMITED.

## BRITISH WEIGHTS AND MEASURES WITH METRICAL EQUIVALENTS.

### LINEAR MEASURE.

Mile.	Furlongs.	Poles.	Yards.	Feet.	Inches.	Metrical Equivalents.
1	8 1	320 40 1	1760 220 5·5 1	5280 660 16·5 3 1	63,360 7,920 198 36 12 1	1609·31 <i>m</i> 201·16 <i>m</i> 5·03 <i>m</i> 91·44 <i>cm</i> 30·48 <i>cm</i> 2·54 <i>cm</i>

### SURVEYING MEASURE (Linear).

Mile.	Furlongs.	Chains.	Poles.	Yards.	Feet.	Links.	Metrical Equivalents.
1	8 1	80 10 1	320 40 4 1	1760 220 22 5·5 1	5280 660 66 16·5 3 1	8000 1000 100 25 4·54 1·51 1	1609·31 <i>m</i> 201·16 <i>m</i> 20·12 <i>m</i> 5·03 <i>m</i> 91·44 <i>cm</i> 30·48 <i>cm</i> 20·12 <i>cm</i>

### SQUARE MEASURE.

Square Mile.	Acres.	Square Chains.	Sq. Poles or Perches.	Square Yards.	Square Feet.	Square Links.	Metrical Equivalents.
1	640 1	6400 10 1	102,400 160 16 1	3,097,600 4,840 484 30·25 1	— 43,560 4,356 272·25 9 1	— 100,000 10,000 625 20·7 2·3 1	258·99 <i>ha</i> 4046·71 <i>m</i> <sup>2</sup> 404·67 <i>m</i> <sup>2</sup> 25·29 <i>m</i> <sup>2</sup> ·836 <i>m</i> <sup>2</sup> 928·99 <i>cm</i> <sup>2</sup> 404·67 <i>cm</i> <sup>2</sup>

# REDPATH, BROWN & CO., LIMITED.

## METRIC UNITS AND BRITISH EQUIVALENTS.

### METRIC ABBREVIATIONS.

Linear Measure.	Square Measure.	Cubic Measure.	Measure of Capacity.	Weight.
<i>km</i> =kilometre <i>m</i> =metre <i>dm</i> =decimetre <i>cm</i> =centimetre <i>mm</i> =millimetre	<i>km<sup>2</sup></i> =sq. kilometre <i>ha</i> =hectare <i>a</i> =are <i>m<sup>2</sup></i> =sq. metre <i>dm<sup>2</sup></i> = " decimetre <i>cm<sup>2</sup></i> = " centimetre <i>mm<sup>2</sup></i> = " millimetre	<i>m<sup>3</sup></i> =cub. metre <i>dm<sup>3</sup></i> = " decimetre <i>cm<sup>3</sup></i> = " centimetre <i>mm<sup>3</sup></i> = " millimetre	<i>kl</i> =kilolitre <i>hl</i> =hectolitre <i>l</i> =litre <i>dl</i> =decilitre <i>cl</i> =centilitre	<i>kgm</i> =kilogramme <i>gm</i> =gramme <i>dgm</i> =decigramme <i>cgm</i> =centigramme <i>mgm</i> =milligramme

### LINEAR MEASURE.

Kilometres.	Metres.	Decimetres.	Centimetres.	Millimetres.	British Equivalents.
1	1000	10,000	100,000	1,000,000	·0214 mile
	1	10	100	1,000	3·281 feet
		1	10	100	·3281 foot
			1	10	·3937 inch
				1	·03937 inch

### SQUARE MEASURE.

Square Kilometres.	Hectares.	Ares.	Square Metres.	Square Decimetres.	Square Centimetres.	Square Millimetres.	British Equivalents.
1	100	10,000	1,000,000	—	—	—	·3861 sq. mile
	1	100	10,000	1,000,000	—	—	·00386 " mile
		1	100	10,000	1,000,000	—	1076 " feet
			1	100	10,000	1,000,000	10·76 " feet
				1	100	10,000	·1076 " feet
					1	100	·1550 " inch
						1	·00155 " inch

# REDPATH, BROWN & CO., LIMITED.

## BRITISH WEIGHTS AND MEASURES WITH METRICAL EQUIVALENTS.

### CUBIC MEASURES.

Cubic Yard.	Cubic Feet.	Cubic Inches.	Metrical Equivalent.
1	27 1	46,656 1,728 1	·764 m <sup>3</sup> ·029 m <sup>3</sup> 16·386 cm <sup>3</sup>

### CAPACITY.

Quarter.	Bushels.	Pecks.	Gallons.	Quarts.	Pints.	Cubic Inches.	Metrical Equivalent.
1	8 1	32 4 1	64 8 2 1	256 32 8 4 1	512 64 16 8 2 1	17,767·6 2,219·7 554·9 277·5 69·4 34·7 1	290·781 l 36·348 l 9·087 l 4·543 l 1·136 l ·568 l 16·386 ml

### AVOIRDUPOIS WEIGHT.

Ton.	Cwts.	Qrs.	Stones.	Lbs.	Ounces.	Drams.	Metrical Equivalent.
1	20 1	80 4 1	160 8 2 1	2,240 112 28 14 1	35,840 1,792 448 224 16 1	573,440 23,672 7,168 3,584 256 16 1	1,016·048 kgm 50·802 kgm 12·700 kgm 6·350 kgm 453·59 gm 28·35 gm 1·77 gm

# REDPATH, BROWN & CO., LIMITED.

## METRIC UNITS AND BRITISH EQUIVALENTS.

### Significance of Prefixes.

kilo	-	-	-	=	1000	deci	-	-	-	=	·1
hecto	-	-	-	=	100	centi	-	-	-	=	·01
deka	-	-	-	=	10	milli	-	-	-	=	·001

### CUBIC MEASURE.

1 cubic metre	=	1,000,000 $cm^3$	=	35·32 cub. feet.
1 cubic centimetre	=		=	0·06103 cub. inches.

### CAPACITY.

1 litre	=	1000 $cm^3$	=	0·03532 cub. feet.
"			=	0·2201 gallons.
1 litre of water	=	1 $kg$	=	2·205 lbs.

### WEIGHT.

1 Tonne	=	1000 $kg$	=	0·9843 ton.
1 Kilogramme	=	1000 $gms$	=	2·205 lbs.
1 gramme	=		=	0·03527 oz.

### EQUIVALENTS OF MOMENTS OF INERTIA AND SECTION MODULI.

Moment of Inertia in inch units	=	·02403	×	Moment of Inertia in $cm$ units.
" " " " $cm$ units	=	41·6198	×	" " " " inch units.
Modulus of Section in inch units	=	·06103	×	Modulus of Section in $cm$ units.
" " " " $cm$ units	=	16·3860	×	" " " " inch units.

# REDPATH, BROWN & CO., LIMITED.

## EQUIVALENTS OF FEET AND INCHES IN MILLIMETRES.

Feet.	0"	1"	2"	3"	4"	5"
0		25.4	50.8	76.2	102	127
1	805	330	356	381	406	432
2	610	635	660	686	711	737
3	914	940	965	991	1016	1041
4	1219	1245	1270	1295	1321	1346
5	1524	1549	1575	1600	1626	1651
6	1829	1854	1880	1905	1930	1956
7	2134	2159	2184	2210	2235	2261
8	2438	2464	2489	2515	2540	2565
9	2743	2769	2794	2819	2845	2870
10	3048	3073	3099	3124	3150	3175
11	3353	3378	3404	3429	3454	3480
12	3658	3683	3708	3734	3759	3785
13	3962	3988	4013	4039	4064	4089
14	4267	4293	4318	4343	4369	4394
15	4572	4597	4623	4648	4674	4699
16	4877	4902	4928	4953	4978	5004
17	5182	5207	5232	5258	5283	5309
18	5486	5512	5537	5563	5588	5613
19	5791	5817	5842	5867	5893	5918
20	6096	6121	6147	6172	6198	6223
21	6401	6426	6452	6477	6502	6528
22	6706	6731	6756	6782	6807	6833
23	7010	7036	7061	7087	7112	7137
24	7315	7341	7366	7391	7417	7442
25	7620	7645	7671	7696	7722	7747
26	7925	7950	7975	8001	8026	8052
27	8230	8255	8280	8306	8331	8357
28	8534	8559	8585	8610	8636	8661
29	8839	8864	8890	8915	8941	8966
30	9144	9169	9195	9220	9246	9271
31	9449	9474	9500	9525	9551	9576
32	9753	9778	9804	9829	9855	9880
33	10058	10083	10109	10134	10160	10185
34	10363	10388	10414	10439	10465	10490
35	10668	10693	10719	10744	10770	10795
36	10973	10998	11024	11049	11074	11100
37	11277	11302	11328	11353	11379	11404
38	11582	11607	11633	11658	11684	11709
39	11887	11912	11938	11963	11989	12014

40 feet = 12192 millimetres.

# REDPATH, BROWN & CO., LIMITED.

## EQUIVALENTS OF FEET AND INCHES IN MILLIMETRES.

6"	7"	8"	9"	10"	11"	Feet.
152	178	203	229	254	279	0
457	483	508	533	559	584	1
762	787	813	838	864	889	2
1067	1092	1118	1143	1168	1194	3
1372	1397	1422	1448	1473	1499	4
1676	1702	1727	1753	1778	1803	5
1981	2007	2032	2057	2083	2108	6
2286	2311	2337	2362	2388	2413	7
2591	2616	2642	2667	2692	2718	8
2896	2921	2946	2972	2997	3023	9
3200	3226	3251	3277	3302	3327	10
3505	3531	3556	3581	3607	3632	11
3810	3835	3861	3886	3912	3937	12
4115	4140	4166	4191	4216	4242	13
4420	4445	4470	4496	4521	4547	14
4724	4750	4775	4801	4826	4851	15
5029	5055	5080	5105	5131	5156	16
5334	5359	5385	5410	5436	5461	17
5639	5664	5690	5715	5740	5766	18
5944	5969	5994	6020	6045	6071	19
6248	6274	6299	6325	6350	6375	20
6553	6579	6604	6629	6655	6680	21
6858	6883	6909	6934	6960	6985	22
7163	7188	7214	7239	7264	7290	23
7467	7493	7518	7543	7569	7594	24
7772	7798	7823	7849	7874	7899	25
8077	8102	8128	8153	8179	8204	26
8382	8407	8433	8458	8484	8509	27
8686	8712	8737	8763	8788	8814	28
8991	9017	9042	9068	9093	9118	29
9296	9322	9347	9373	9398	9423	30
9601	9627	9652	9677	9703	9728	31
9905	9931	9956	9982	10007	10032	32
10210	10236	10261	10287	10312	10337	33
10515	10541	10566	10592	10617	10642	34
10820	10846	10871	10896	10922	10947	35
11125	11151	11176	11201	11227	11252	36
11429	11455	11480	11506	11531	11556	37
11734	11760	11785	11811	11836	11861	38
12039	12065	12090	12116	12141	12166	39

40 feet = 12192 millimetres.

# REDPATH, BROWN & CO., LIMITED.

## EQUIVALENTS OF INCHES IN MILLIMETRES.

Rising by 32nds of an inch to 13 inches.

Inches.	0	$\frac{1}{32}$	$\frac{1}{16}$	$\frac{3}{32}$	$\frac{1}{8}$	$\frac{5}{32}$	$\frac{3}{16}$	$\frac{7}{16}$
0		794	1587	2381	3175	3969	4763	5556
1	25'400	26'193	26'987	27'781	28'574	29'368	30'162	30'956
2	50'799	51'593	52'387	53'180	53'974	54'768	55'561	56'355
3	76'199	76'992	77'786	78'580	79'374	80'167	80'961	81'755
4	101'60	102'39	103'19	103'98	104'77	105'57	106'36	107'15
5	127'00	127'79	128'59	129'38	130'17	130'97	131'76	132'55
6	152'40	153'19	153'98	154'78	155'57	156'37	157'16	157'95
7	177'80	178'59	179'38	180'18	180'97	181'76	182'56	183'35
8	203'20	203'99	204'78	205'58	206'37	207'16	207'96	208'75
9	228'60	229'39	230'18	230'98	231'77	232'56	233'36	234'15
10	254'00	254'79	255'58	256'38	257'17	257'96	258'76	259'55
11	279'39	280'19	280'98	281'78	282'57	283'36	284'16	284'95
12	304'79	305'59	306'38	307'18	307'97	308'76	309'56	310'35

Inches.	$\frac{1}{2}$	$\frac{17}{32}$	$\frac{9}{16}$	$\frac{13}{16}$	$\frac{5}{8}$	$\frac{31}{32}$	$1\frac{1}{16}$	$1\frac{3}{16}$
0	12'700	13'494	14'287	15'081	15'875	16'668	17'463	18'256
1	38'099	38'893	39'687	40'481	41'274	42'068	42'862	43'656
2	63'499	64'293	65'086	65'880	66'674	67'468	68'261	69'055
3	88'898	89'692	90'486	91'280	92'073	92'867	93'661	94'455
4	114'30	115'09	115'89	116'68	117'47	118'27	119'06	119'85
5	139'70	140'49	141'28	142'08	142'87	143'67	144'46	145'25
6	165'10	165'89	166'68	167'48	168'27	169'07	169'86	170'65
7	190'50	191'29	192'08	192'88	193'67	194'47	195'26	196'05
8	215'90	216'69	217'48	218'28	219'07	219'87	220'66	221'45
9	241'30	242'09	242'88	243'68	244'47	245'26	246'06	246'85
10	266'70	267'49	268'28	269'08	269'87	270'66	271'46	272'25
11	292'10	292'89	293'68	294'48	295'27	296'06	296'86	297'65
12	317'49	318'29	319'08	319'88	320'67	321'46	322'26	323'05

13 inches = 330'19 millimetres.

# REDPATH, BROWN & CO., LIMITED.

## EQUIVALENTS OF INCHES IN MILLIMETRES.

Rising by 32nds of an inch to 13 inches.

$\frac{1}{8}$	$\frac{3}{16}$	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{5}{8}$	Inches.
0.353	7.144	7.937	8.731	9.525	10.319	11.112	11.906	0
31.749	32.543	33.337	34.131	34.924	35.718	36.512	37.306	1
57.149	57.943	58.736	59.530	60.324	61.118	61.911	62.705	2
82.549	83.342	84.136	84.930	85.723	86.517	87.311	88.105	3
107.95	108.74	109.54	110.33	111.12	111.92	112.71	113.50	4
133.35	134.14	134.94	135.73	136.52	137.32	138.11	138.90	5
158.75	159.54	160.33	161.13	161.92	162.72	163.51	164.30	6
184.15	184.94	185.73	186.53	187.32	188.11	188.91	189.70	7
209.55	210.34	211.13	211.93	212.72	213.51	214.31	215.10	8
234.95	235.74	236.53	237.33	238.12	238.91	239.71	240.50	9
260.35	261.14	261.93	262.73	263.52	264.31	265.11	265.90	10
285.74	286.54	287.33	288.13	288.92	289.71	290.51	291.30	11
311.14	311.94	312.73	313.53	314.32	315.11	315.91	316.70	12

$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	$1$	$1\frac{1}{8}$	Inches.
19.050	19.843	20.637	21.431	22.225	23.019	23.812	24.606	0
44.449	45.243	46.037	46.830	47.624	48.418	49.212	50.006	1
69.849	70.642	71.436	72.230	73.024	73.817	74.611	75.405	2
95.248	96.042	96.836	97.629	98.423	99.217	100.01	100.80	3
120.65	121.44	122.24	123.03	123.82	124.62	125.41	126.20	4
146.05	146.84	147.63	148.43	149.22	150.02	150.81	151.60	5
171.45	172.24	173.03	173.83	174.62	175.42	176.21	177.00	6
196.85	197.64	198.43	199.23	200.02	200.82	201.61	202.40	7
222.25	223.04	223.83	224.63	225.42	226.21	227.01	227.80	8
247.65	248.44	249.23	250.03	250.82	251.61	252.41	253.20	9
273.05	273.84	274.63	275.43	276.22	277.01	277.81	278.60	10
298.44	299.24	300.03	300.83	301.62	302.41	303.21	304.00	11
323.84	324.64	325.43	326.23	327.02	327.81	328.61	329.40	12

13 inches = 330.19 millimetres.

# REDPATH, BROWN & CO., LIMITED.

## EQUIVALENTS OF MILLIMETRES IN INCHES.

Milli- metres.	Inches.	Milli- metres.	Inches.	Milli- metres.	Inches.	Milli- metres.	Inches.	Milli- metres.	Inches.
1	.039	51	2.008	101	3.976	151	5.945	201	7.913
2	.079	52	2.047	102	4.016	152	5.984	202	7.953
3	.118	53	2.087	103	4.055	153	6.024	203	7.992
4	.157	54	2.126	104	4.095	154	6.063	204	8.032
5	.197	55	2.165	105	4.134	155	6.102	205	8.071
6	.236	56	2.205	106	4.173	156	6.142	206	8.110
7	.276	57	2.244	107	4.213	157	6.181	207	8.150
8	.315	58	2.283	108	4.252	158	6.221	208	8.189
9	.354	59	2.323	109	4.291	159	6.260	209	8.228
10	.394	60	2.362	110	4.331	160	6.299	210	8.268
11	.433	61	2.402	111	4.370	161	6.339	211	8.307
12	.472	62	2.441	112	4.409	162	6.378	212	8.347
13	.512	63	2.480	113	4.449	163	6.417	213	8.386
14	.551	64	2.520	114	4.488	164	6.457	214	8.425
15	.591	65	2.559	115	4.528	165	6.496	215	8.465
16	.630	66	2.598	116	4.567	166	6.535	216	8.504
17	.669	67	2.638	117	4.606	167	6.575	217	8.543
18	.709	68	2.677	118	4.646	168	6.614	218	8.583
19	.748	69	2.717	119	4.685	169	6.654	219	8.622
20	.787	70	2.756	120	4.724	170	6.693	220	8.661
21	.827	71	2.795	121	4.764	171	6.732	221	8.701
22	.866	72	2.835	122	4.803	172	6.772	222	8.740
23	.906	73	2.874	123	4.843	173	6.811	223	8.780
24	.945	74	2.913	124	4.882	174	6.850	224	8.819
25	.984	75	2.953	125	4.921	175	6.890	225	8.858
26	1.024	76	2.992	126	4.961	176	6.929	226	8.898
27	1.063	77	3.032	127	5.000	177	6.969	227	8.937
28	1.102	78	3.071	128	5.039	178	7.008	228	8.976
29	1.142	79	3.110	129	5.079	179	7.047	229	9.016
30	1.181	80	3.150	130	5.118	180	7.087	230	9.055
31	1.220	81	3.189	131	5.158	181	7.126	231	9.095
32	1.260	82	3.228	132	5.197	182	7.166	232	9.134
33	1.299	83	3.268	133	5.236	183	7.205	233	9.173
34	1.339	84	3.307	134	5.276	184	7.244	234	9.213
35	1.378	85	3.346	135	5.315	185	7.284	235	9.252
36	1.417	86	3.386	136	5.354	186	7.323	236	9.291
37	1.457	87	3.425	137	5.394	187	7.362	237	9.331
38	1.496	88	3.465	138	5.433	188	7.402	238	9.370
39	1.535	89	3.504	139	5.472	189	7.441	239	9.410
40	1.575	90	3.543	140	5.512	190	7.480	240	9.449
41	1.614	91	3.583	141	5.551	191	7.520	241	9.488
42	1.654	92	3.622	142	5.591	192	7.559	242	9.528
43	1.693	93	3.661	143	5.630	193	7.598	243	9.567
44	1.732	94	3.701	144	5.669	194	7.638	244	9.606
45	1.772	95	3.740	145	5.709	195	7.677	245	9.646
46	1.811	96	3.780	146	5.748	196	7.717	246	9.685
47	1.850	97	3.819	147	5.787	197	7.756	247	9.724
48	1.890	98	3.858	148	5.827	198	7.795	248	9.764
49	1.929	99	3.898	149	5.866	199	7.835	249	9.803
50	1.969	100	3.937	150	5.906	200	7.874	250	9.843

# REDPATH, BROWN & CO., LIMITED.

## EQUIVALENTS OF MILLIMETRES IN INCHES.

Milli- metres.	Inches.	Milli- metres.	Inches.	Milli- metres.	Inches.	Milli- metres.	Inches.	Milli- metres.	Inches.
251	9.882	301	11.850	351	13.819	401	15.788	451	17.756
252	9.921	302	11.890	352	13.858	402	15.827	452	17.795
253	9.961	303	11.929	353	13.898	403	15.866	453	17.835
254	10.000	304	11.969	354	13.937	404	15.906	454	17.874
255	10.039	305	12.008	355	13.977	405	15.945	455	17.914
256	10.079	306	12.047	356	14.016	406	15.984	456	17.953
257	10.118	307	12.087	357	14.056	407	16.024	457	17.992
258	10.158	308	12.126	358	14.095	408	16.063	458	18.032
259	10.197	309	12.165	359	14.134	409	16.103	459	18.071
260	10.236	310	12.205	360	14.173	410	16.142	460	18.110
261	10.276	311	12.244	361	14.213	411	16.181	461	18.150
262	10.315	312	12.284	362	14.252	412	16.221	462	18.189
263	10.354	313	12.323	363	14.291	413	16.260	463	18.229
264	10.394	314	12.362	364	14.331	414	16.299	464	18.268
265	10.433	315	12.402	365	14.370	415	16.339	465	18.307
266	10.473	316	12.441	366	14.410	416	16.378	466	18.347
267	10.512	317	12.480	367	14.449	417	16.417	467	18.386
268	10.551	318	12.520	368	14.488	418	16.457	468	18.425
269	10.591	319	12.559	369	14.528	419	16.496	469	18.465
270	10.630	320	12.599	370	14.567	420	16.536	470	18.504
271	10.669	321	12.638	371	14.606	421	16.575	471	18.543
272	10.709	322	12.677	372	14.646	422	16.614	472	18.583
273	10.748	323	12.717	373	14.685	423	16.654	473	18.622
274	10.787	324	12.756	374	14.725	424	16.693	474	18.662
275	10.827	325	12.795	375	14.764	425	16.732	475	18.701
276	10.866	326	12.835	376	14.803	426	16.772	476	18.740
277	10.906	327	12.874	377	14.843	427	16.811	477	18.780
278	10.945	328	12.913	378	14.882	428	16.851	478	18.819
279	10.984	329	12.953	379	14.921	429	16.890	479	18.858
280	11.024	330	12.992	380	14.961	430	16.929	480	18.898
281	11.063	331	13.032	381	15.000	431	16.969	481	18.937
282	11.102	332	13.071	382	15.040	432	17.008	482	18.977
283	11.142	333	13.110	383	15.079	433	17.047	483	19.016
284	11.181	334	13.150	384	15.118	434	17.087	484	19.056
285	11.221	335	13.189	385	15.158	435	17.126	485	19.096
286	11.260	336	13.228	386	15.197	436	17.166	486	19.134
287	11.299	337	13.268	387	15.236	437	17.205	487	19.173
288	11.339	338	13.307	388	15.276	438	17.244	488	19.213
289	11.378	339	13.347	389	15.315	439	17.284	489	19.252
290	11.417	340	13.386	390	15.354	440	17.323	490	19.292
291	11.457	341	13.425	391	15.394	441	17.362	491	19.331
292	11.496	342	13.465	392	15.433	442	17.402	492	19.370
293	11.536	343	13.504	393	15.473	443	17.441	493	19.410
294	11.575	344	13.543	394	15.512	444	17.480	494	19.449
295	11.614	345	13.583	395	15.551	445	17.520	495	19.488
296	11.654	346	13.622	396	15.591	446	17.559	496	19.528
297	11.693	347	13.662	397	15.630	447	17.599	497	19.567
298	11.732	348	13.701	398	15.669	448	17.638	498	19.606
299	11.772	349	13.740	399	15.709	449	17.677	499	19.646
300	11.811	350	13.780	400	15.748	450	17.717	500	19.685

# REDPATH, BROWN & CO., LIMITED.

## EQUIVALENTS OF MILLIMETRES IN INCHES.

Milli- metres.	Inches.	Milli- metres.	Inches.	Milli- metres.	Inches.	Milli- metres.	Inches.	Milli- metres.	Inches.
501	19.725	551	21.693	601	23.662	651	25.630	701	27.599
502	19.764	552	21.732	602	23.701	652	25.670	702	27.638
503	19.803	553	21.772	603	23.740	653	25.709	703	27.677
504	19.843	554	21.811	604	23.780	654	25.748	704	27.717
505	19.882	555	21.851	605	23.819	655	25.788	705	27.756
506	19.921	556	21.890	606	23.858	656	25.827	706	27.796
507	19.961	557	21.929	607	23.898	657	25.866	707	27.835
508	20.000	558	21.969	608	23.937	658	25.906	708	27.874
509	20.040	559	22.008	609	23.977	659	25.945	709	27.914
510	20.079	560	22.047	610	24.016	660	25.984	710	27.953
511	20.118	561	22.087	611	24.055	661	26.024	711	27.992
512	20.158	562	22.126	612	24.095	662	26.063	712	28.032
513	20.197	563	22.166	613	24.134	663	26.103	713	28.071
514	20.236	564	22.205	614	24.173	664	26.142	714	28.110
515	20.276	565	22.244	615	24.213	665	26.181	715	28.150
516	20.315	566	22.284	616	24.252	666	26.221	716	28.189
517	20.355	567	22.323	617	24.292	667	26.260	717	28.229
518	20.394	568	22.362	618	24.331	668	26.299	718	28.268
519	20.433	569	22.402	619	24.370	669	26.339	719	28.307
520	20.473	570	22.441	620	24.410	670	26.378	720	28.347
521	20.512	571	22.481	621	24.449	671	26.418	721	28.386
522	20.551	572	22.520	622	24.488	672	26.457	722	28.425
523	20.591	573	22.559	623	24.528	673	26.496	723	28.465
524	20.630	574	22.599	624	24.567	674	26.536	724	28.504
525	20.669	575	22.638	625	24.607	675	26.575	725	28.544
526	20.709	576	22.677	626	24.646	676	26.614	726	28.583
527	20.748	577	22.717	627	24.685	677	26.654	727	28.622
528	20.788	578	22.756	628	24.725	678	26.693	728	28.662
529	20.827	579	22.795	629	24.764	679	26.733	729	28.701
530	20.866	580	22.835	630	24.803	680	26.772	730	28.740
531	20.906	581	22.874	631	24.843	681	26.811	731	28.780
532	20.945	582	22.914	632	24.882	682	26.851	732	28.819
533	20.984	583	22.953	633	24.921	683	26.890	733	28.859
534	21.024	584	22.992	634	24.961	684	26.929	734	28.898
535	21.063	585	23.032	635	25.000	685	26.969	735	28.937
536	21.103	586	23.071	636	25.040	686	27.008	736	28.977
537	21.142	587	23.110	637	25.079	687	27.047	737	29.016
538	21.181	588	23.150	638	25.118	688	27.087	738	29.056
539	21.221	589	23.189	639	25.158	689	27.126	739	29.095
540	21.260	590	23.229	640	25.197	690	27.166	740	29.134
541	21.299	591	23.268	641	25.236	691	27.205	741	29.173
542	21.339	592	23.307	642	25.276	692	27.244	742	29.213
543	21.378	593	23.347	643	25.315	693	27.284	743	29.252
544	21.418	594	23.386	644	25.355	694	27.323	744	29.292
545	21.457	595	23.425	645	25.394	695	27.362	745	29.331
546	21.496	596	23.464	646	25.433	696	27.402	746	29.370
547	21.536	597	23.503	647	25.473	697	27.441	747	29.410
548	21.575	598	23.543	648	25.512	698	27.481	748	29.449
549	21.614	599	23.582	649	25.551	699	27.520	749	29.488
550	21.654	600	23.622	650	25.591	700	27.559	750	29.528

# REDPATH, BROWN & CO., LIMITED.

## EQUIVALENTS OF MILLIMETRES IN INCHES.

Milli- metres.	Inches.	Milli- metres.	Inches.	Milli- metres.	Inches.	Milli- metres.	Inches.	Milli- metres.	Inches.
751	29.567	801	31.538	851	33.504	901	35.478	951	37.441
752	29.567	802	31.576	852	33.544	902	35.512	952	37.481
753	29.568	803	31.614	853	33.583	903	35.552	953	37.520
754	29.585	804	31.654	854	33.622	904	35.591	954	37.559
755	29.725	805	31.693	855	33.662	905	35.630	955	37.599
756	29.704	806	31.733	856	33.701	906	35.670	956	37.638
757	29.803	807	31.772	857	33.740	907	35.709	957	37.677
758	29.843	808	31.811	858	33.780	908	35.748	958	37.717
759	29.882	809	31.851	859	33.819	909	35.788	959	37.756
760	29.922	810	31.890	860	33.859	910	35.827	960	37.796
761	29.961	811	31.929	861	33.898	911	35.866	961	37.835
762	30.000	812	31.969	862	33.937	912	35.906	962	37.874
763	30.040	813	32.008	863	33.977	913	35.945	963	37.914
764	30.079	814	32.048	864	34.016	914	35.985	964	37.953
765	30.118	815	32.087	865	34.055	915	36.024	965	37.992
766	30.158	816	32.126	866	34.095	916	36.063	966	38.032
767	30.197	817	32.166	867	34.134	917	36.103	967	38.071
768	30.236	818	32.205	868	34.174	918	36.142	968	38.111
769	30.276	819	32.244	869	34.213	919	36.181	969	38.150
770	30.315	820	32.284	870	34.252	920	36.221	970	38.189
771	30.355	821	32.323	871	34.292	921	36.260	971	38.229
772	30.394	822	32.362	872	34.331	922	36.300	972	38.268
773	30.433	823	32.402	873	34.370	923	36.339	973	38.307
774	30.473	824	32.441	874	34.410	924	36.378	974	38.347
775	30.512	825	32.481	875	34.449	925	36.418	975	38.386
776	30.551	826	32.520	876	34.488	926	36.457	976	38.426
777	30.591	827	32.559	877	34.528	927	36.496	977	38.465
778	30.630	828	32.599	878	34.567	928	36.536	978	38.504
779	30.670	829	32.638	879	34.607	929	36.575	979	38.544
780	30.709	830	32.677	880	34.646	930	36.615	980	38.583
781	30.748	831	32.717	881	34.685	931	36.654	981	38.622
782	30.788	832	32.756	882	34.725	932	36.693	982	38.662
783	30.827	833	32.796	883	34.764	933	36.733	983	38.701
784	30.866	834	32.835	884	34.803	934	36.772	984	38.741
785	30.906	835	32.874	885	34.843	935	36.811	985	38.780
786	30.945	836	32.914	886	34.882	936	36.851	986	38.819
787	30.985	837	32.953	887	34.922	937	36.890	987	38.859
788	31.024	838	32.992	888	34.961	938	36.929	988	38.898
789	31.063	839	33.032	889	35.000	939	36.969	989	38.937
790	31.103	840	33.071	890	35.040	940	37.008	990	38.977
791	31.142	841	33.111	891	35.079	941	37.048	991	39.016
792	31.181	842	33.150	892	35.118	942	37.087	992	39.055
793	31.221	843	33.189	893	35.158	943	37.126	993	39.095
794	31.260	844	33.229	894	35.197	944	37.166	994	39.134
795	31.299	845	33.268	895	35.237	945	37.205	995	39.174
796	31.339	846	33.307	896	35.276	946	37.244	996	39.213
797	31.378	847	33.347	897	35.316	947	37.284	997	39.252
798	31.418	848	33.386	898	35.355	948	37.323	998	39.292
799	31.457	849	33.425	899	35.394	949	37.363	999	39.331
800	31.496	850	33.465	900	35.433	950	37.402	1000	39.370

# REDPATH, BROWN & CO., LIMITED.

## EQUIVALENTS OF FEET IN METRES.

1 Foot = '3048 of 1 Metre.

Feet.	0	1	2	3	4	5	6	7	8	9
1	'3048	'3353	'3657	'3962	'4267	'4572	'4877	'5181	'5486	'5791
2	'0096	'6401	'6705	'7010	'7315	'7620	'7925	'8229	'8534	'8839
3	'9144	'0449	'0753	'1058	'1363	'1668	'1973	'1277	'1582	'1887
4	1'2192	1'2497	1'2801	1'3106	1'3411	1'3716	1'4020	1'4325	1'4630	1'4935
5	1'5240	1'5544	1'5849	1'6154	1'6459	1'6764	1'7068	1'7373	1'7678	1'7983
6	1'8288	1'8592	1'8897	1'9202	1'9507	1'9812	2'0116	2'0421	2'0726	2'1031
7	2'1336	2'1640	2'1945	2'2250	2'2555	2'2860	2'3164	2'3469	2'3774	2'4079
8	2'4384	2'4688	2'4993	2'5298	2'5603	2'5907	2'6212	2'6517	2'6822	2'7127
9	2'7431	2'7736	2'8041	2'8346	2'8651	2'8955	2'9260	2'9565	2'9870	3'0175
10	3'0479	3'0784	3'1089	3'1394	3'1699	3'2003	3'2308	3'2613	3'2918	3'3223

## EQUIVALENT OF SQUARE INCHES IN SQUARE CENTIMETRES.

1 Square Inch = 6'4514 Square Centimetres.

Square Inches	0	1	2	3	4	5	6	7	8	9
1	6'451	7'090	7'742	8'387	9'032	9'677	10'322	10'967	11'612	12'258
2	12'908	13'548	14'193	14'838	15'483	16'128	16'774	17'419	18'064	18'709
3	19'354	19'999	20'644	21'289	21'935	22'580	23'225	23'870	24'515	25'160
4	25'805	26'451	27'096	27'741	28'386	29'031	29'676	30'321	30'967	31'612
5	32'257	32'902	33'547	34'192	34'837	35'482	36'128	36'773	37'418	38'063
6	38'708	39'353	39'998	40'644	41'289	41'934	42'579	43'224	43'869	44'514
7	45'160	45'805	46'450	47'095	47'740	48'385	49'030	49'675	50'321	50'966
8	51'611	52'256	52'901	53'546	54'191	54'837	55'482	56'127	56'772	57'417
9	58'062	58'707	59'353	59'998	60'643	61'288	61'933	62'578	63'223	63'868
10	64'514	65'159	65'804	66'449	67'094	67'739	68'384	69'030	69'675	70'320

## EQUIVALENTS OF SQUARE FEET IN SQUARE METRES.

1 Square Foot = '0929 of 1 Square Metre.

Square Feet.	0	1	2	3	4	5	6	7	8	9
1	'0929	'1022	'1115	'1208	'1301	'1393	'1486	'1579	'1672	'1765
2	'1858	'1951	'2044	'2137	'2230	'2322	'2415	'2508	'2601	'2694
3	'2787	'2880	'2973	'3066	'3159	'3251	'3344	'3437	'3530	'3623
4	'3716	'3809	'3902	'3995	'4088	'4180	'4273	'4366	'4459	'4552
5	'4645	'4738	'4831	'4924	'5017	'5109	'5202	'5295	'5388	'5481
6	'5574	'5667	'5760	'5853	'5946	'6038	'6131	'6224	'6317	'6410
7	'6503	'6596	'6689	'6782	'6875	'6967	'7060	'7153	'7246	'7339
8	'7432	'7525	'7618	'7711	'7804	'7896	'7989	'8082	'8175	'8268
9	'8361	'8454	'8547	'8640	'8733	'8825	'8918	'9011	'9104	'9197
10	'9290	'9383	'9476	'9569	'9662	'9754	'9847	'9940	1'0033	1'0126

# REDPATH, BROWN & CO., LIMITED.

## EQUIVALENTS OF METRES IN FEET.

1 Metre = 3.2809 Feet.

Metres.	0	1	2	3	4	5	6	7	8	9
1	3.2809	3.6090	3.9371	4.2652	4.5933	4.9213	5.2494	5.5775	5.9056	6.2337
2	6.5618	6.8899	7.2180	7.5461	7.8742	8.2022	8.5303	8.8584	9.1865	9.5146
3	9.8427	10.1708	10.4989	10.8270	11.1551	11.4831	11.8112	12.1393	12.4674	12.7955
4	13.1236	13.4517	13.7798	14.1079	14.4360	14.7640	15.0921	15.4202	15.7483	16.0764
5	16.4045	16.7326	17.0607	17.3888	17.7169	18.0449	18.3730	18.7011	19.0292	19.3573
6	19.6854	20.0135	20.3416	20.6697	20.9978	21.3258	21.6539	21.9820	22.3101	22.6382
7	22.9663	23.2944	23.6225	23.9506	24.2787	24.6067	24.9348	25.2629	25.5910	25.9191
8	26.2472	26.5753	26.9034	27.2315	27.5596	27.8876	28.2157	28.5438	28.8719	29.2000
9	29.5281	29.8562	30.1843	30.5124	30.8405	31.1685	31.4966	31.8247	32.1528	32.4809
10	32.8090	33.1371	33.4652	33.7933	34.1213	34.4494	34.7775	35.1056	35.4337	35.7618

## EQUIVALENTS OF SQUARE CENTIMETRES IN SQUARE INCHES.

1 Square Centimetre = .1550 of 1 Square Inch.

Square Centimetres.	0	1	2	3	4	5	6	7	8	9
1	.1550	.1705	.1860	.2015	.2170	.2325	.2480	.2635	.2790	.2945
2	.3100	.3255	.3410	.3565	.3720	.3875	.4030	.4185	.4340	.4495
3	.4650	.4805	.4960	.5115	.5270	.5425	.5580	.5735	.5890	.6045
4	.6200	.6355	.6510	.6665	.6820	.6975	.7130	.7285	.7440	.7595
5	.7750	.7905	.8060	.8215	.8370	.8525	.8680	.8835	.8990	.9145
6	.9300	.9455	.9610	.9765	.9920	1.0075	1.0230	1.0385	1.0540	1.0695
7	1.0850	1.1005	1.1160	1.1315	1.1470	1.1625	1.1780	1.1935	1.2090	1.2245
8	1.2400	1.2555	1.2710	1.2865	1.3020	1.3175	1.3330	1.3485	1.3640	1.3795
9	1.3950	1.4105	1.4260	1.4415	1.4570	1.4725	1.4880	1.5035	1.5190	1.5345
10	1.5500	1.5655	1.5810	1.5965	1.6120	1.6275	1.6430	1.6585	1.6740	1.6895

## EQUIVALENTS OF SQUARE METRES IN SQUARE FEET.

1 Square Metre = 10.7643 Square Feet.

Square Metres.	0	1	2	3	4	5	6	7	8	9
1	10.764	11.841	12.917	13.994	15.070	16.146	17.223	18.299	19.376	20.452
2	21.529	22.605	23.681	24.758	25.834	26.911	27.987	29.064	30.140	31.216
3	32.293	33.369	34.446	35.522	36.599	37.675	38.751	39.828	40.904	41.981
4	43.057	44.134	45.210	46.286	47.363	48.439	49.516	50.592	51.669	52.745
5	53.821	54.898	55.974	57.051	58.127	59.204	60.280	61.356	62.433	63.509
6	64.586	65.662	66.739	67.815	68.892	69.968	71.044	72.121	73.197	74.274
7	75.350	76.427	77.503	78.579	79.656	80.732	81.809	82.885	83.962	85.038
8	86.114	87.191	88.267	89.344	90.420	91.497	92.573	93.649	94.726	95.802
9	96.879	97.955	99.032	100.108	101.184	102.261	103.337	104.414	105.490	106.567
10	107.643	108.719	109.796	110.872	111.949	113.025	114.102	115.178	116.254	117.331

# REDPATH, BROWN & CO., LIMITED.

## EQUIVALENTS OF POUNDS IN KILOGRAMMES.

1 Pound = '45359 of 1 Kilogramme.

Pounds.	0	1	2	3	4	5	6	7	8	9
1	'4536	'4989	'5443	'5897	'6350	'6804	'7257	'7711	'8165	'8618
2	'9072	'9525	'9979	1'0433	1'0886	1'1340	1'1793	1'2247	1'2701	1'3154
3	1'3608	1'4061	1'4515	1'4969	1'5422	1'5876	1'6329	1'6783	1'7236	1'7690
4	1'8144	1'8597	1'9051	1'9504	1'9958	2'0412	2'0865	2'1319	2'1772	2'2226
5	2'2680	2'3133	2'3587	2'4040	2'4494	2'4948	2'5401	2'5855	2'6308	2'6762
6	2'7216	2'7669	2'8123	2'8576	2'9030	2'9483	2'9937	3'0391	3'0844	3'1298
7	3'1751	3'2205	3'2659	3'3112	3'3566	3'4019	3'4473	3'4927	3'5380	3'5834
8	3'6287	3'6741	3'7195	3'7648	3'8102	3'8555	3'9009	3'9463	3'9916	4'0370
9	4'0823	4'1277	4'1730	4'2184	4'2638	4'3091	4'3545	4'3998	4'4452	4'4906
10	4'5359	4'5813	4'6266	4'6720	4'7174	4'7627	4'8081	4'8534	4'8988	4'9442

## EQUIVALENTS OF POUNDS PER SQUARE INCH IN KILOGRAMMES PER SQUARE CENTIMETRE.

1 Pound per Square Inch = '07031 of 1 Kilogramme per Square Centimetre.

Pounds.	0	1	2	3	4	5	6	7	8	9
1	'0703	'0773	'0844	'0914	'0984	'1055	'1125	'1195	'1266	'1336
2	'1406	'1476	'1547	'1617	'1687	'1758	'1828	'1898	'1969	'2039
3	'2109	'2180	'2250	'2320	'2390	'2461	'2531	'2601	'2672	'2742
4	'2812	'2883	'2953	'3023	'3094	'3164	'3234	'3305	'3375	'3446
5	'3516	'3586	'3656	'3726	'3797	'3867	'3937	'4008	'4078	'4148
6	'4219	'4289	'4359	'4429	'4500	'4570	'4640	'4711	'4781	'4851
7	'4922	'4992	'5062	'5133	'5203	'5273	'5344	'5414	'5484	'5554
8	'5625	'5695	'5765	'5836	'5906	'5976	'6047	'6117	'6187	'6258
9	'6328	'6398	'6468	'6539	'6609	'6679	'6750	'6820	'6890	'6961
10	'7031	'7101	'7172	'7242	'7312	'7382	'7453	'7523	'7593	'7664

## EQUIVALENTS OF POUNDS PER FOOT IN KILOGRAMMES PER METRE.

1 Pound per Foot = 1'48819 Kilogrammes per Metre.

Pounds per Foot.	0	1	2	3	4	5	6	7	8	9
1	1'4882	1'6370	1'7858	1'9346	2'0835	2'2323	2'3811	2'5299	2'6787	2'8276
2	2'9764	3'1252	3'2740	3'4228	3'5717	3'7205	3'8693	4'0181	4'1669	4'3158
3	4'4646	4'6134	4'7622	4'9110	5'0599	5'2087	5'3575	5'5063	5'6551	5'8039
4	5'9528	6'1016	6'2504	6'3992	6'5480	6'6969	6'8457	6'9945	7'1433	7'2921
5	7'4410	7'5898	7'7386	7'8874	8'0362	8'1851	8'3339	8'4827	8'6315	8'7803
6	8'9292	9'0780	9'2268	9'3756	9'5244	9'6732	9'8221	9'9709	10'1197	10'2685
7	10'4178	10'5662	10'7150	10'8638	11'0126	11'1614	11'3103	11'4591	11'6079	11'7567
8	11'9055	12'0544	12'2032	12'3520	12'5008	12'6496	12'7984	12'9472	13'0961	13'2449
9	13'3937	13'5425	13'6914	13'8402	13'9890	14'1378	14'2866	14'4355	14'5843	14'7331
10	14'8819	15'0307	15'1796	15'3284	15'4772	15'6260	15'7748	15'9237	16'0725	16'2213

# REDPATH, BROWN & CO., LIMITED.

## EQUIVALENTS OF KILOGRAMMES IN POUNDS.

1 Kilogramme = 2.2046 Pounds.

Kilogrammes.	0	1	2	3	4	5	6	7	8	9
1	2.2046	2.4251	2.6455	2.8660	3.0865	3.3069	3.5274	3.7479	3.9683	4.1888
2	4.4092	4.6297	4.8502	5.0706	5.2911	5.5116	5.7320	5.9525	6.1729	6.3934
3	6.6139	6.8343	7.0548	7.2752	7.4957	7.7162	7.9366	8.1571	8.3776	8.5980
4	8.8185	9.0389	9.2594	9.4799	9.7003	9.9208	10.1413	10.3617	10.5822	10.8026
5	11.0231	11.2436	11.4640	11.6845	11.9050	12.1254	12.3459	12.5663	12.7868	13.0073
6	13.2277	13.4482	13.6687	13.8891	14.1096	14.3300	14.5505	14.7710	14.9914	15.2119
7	15.4323	15.6528	15.8733	16.0937	16.3142	16.5347	16.7551	16.9756	17.1960	17.4165
8	17.6370	17.8574	18.0779	18.2984	18.5188	18.7393	18.9597	19.1802	19.4007	19.6211
9	19.8416	20.0621	20.2825	20.5030	20.7234	20.9439	21.1644	21.3848	21.6053	21.8258
10	22.0462	22.2667	22.4871	22.7076	22.9281	23.1485	23.3690	23.5894	23.8099	24.0304

## EQUIVALENTS OF KILOGRAMMES PER SQUARE CENTIMETRE IN POUNDS PER SQUARE INCH.

1 Kilogramme per Square Centimetre = 14.2233 Pounds per Square Inch.

Kilogrammes per Square Centimetre.	0	1	2	3	4	5	6	7	8	9
1	14.223	15.646	17.067	18.490	19.912	21.334	22.757	24.179	25.601	27.023
2	28.446	29.868	31.290	32.712	34.135	35.557	36.979	38.402	39.824	41.246
3	42.668	44.091	45.513	46.935	48.358	49.780	51.202	52.624	54.047	55.469
4	56.891	58.314	59.736	61.158	62.580	64.003	65.425	66.847	68.270	69.692
5	71.114	72.536	73.959	75.381	76.803	78.226	79.648	81.070	82.492	83.915
6	85.337	86.759	88.181	89.604	91.026	92.448	93.871	95.293	96.715	98.137
7	99.560	100.982	102.404	103.827	105.249	106.671	108.093	109.516	110.938	112.360
8	113.783	115.205	116.627	118.049	119.472	120.894	122.316	123.739	125.161	126.583
9	128.005	129.428	130.850	132.272	133.695	135.117	136.539	137.961	139.384	140.806
10	142.228	143.650	145.073	146.495	147.917	149.340	150.762	152.184	153.606	155.029

## EQUIVALENTS OF KILOGRAMMES PER METRE IN POUNDS PER FOOT.

1 Kilogramme per Metre = 3.7196 of 1 Pound per Foot.

Kilogrammes per Metre.	0	1	2	3	4	5	6	7	8	9
1	3.720	3.7892	3.8583	3.9275	3.9967	4.0659	4.1351	4.2043	4.2735	4.3427
2	7.440	7.5784	7.7167	7.8550	7.9933	8.1316	8.2699	8.4082	8.5465	8.6848
3	11.160	11.3576	11.5552	11.7528	11.9504	12.1480	12.3456	12.5432	12.7408	12.9384
4	14.880	15.1360	15.3920	15.6480	15.9040	16.1600	16.4160	16.6720	16.9280	17.1840
5	18.600	18.9120	19.2240	19.5360	19.8480	20.1600	20.4720	20.7840	21.0960	21.4080
6	22.320	22.6800	23.0400	23.4000	23.7600	24.1200	24.4800	24.8400	25.2000	25.5600
7	26.040	26.4480	26.8560	27.2640	27.6720	28.0800	28.4880	28.8960	29.3040	29.7120
8	29.760	30.2160	30.6720	31.1280	31.5840	32.0400	32.4960	32.9520	33.4080	33.8640
9	33.480	33.9840	34.4880	34.9920	35.4960	35.9520	36.4080	36.8640	37.3200	37.7760
10	37.840	38.3920	38.9440	39.4960	40.0480	40.6000	41.1520	41.7040	42.2560	42.8080



REDPATH, BROWN & CO., LIMITED.

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**PART V.**  
**DETAILS.**

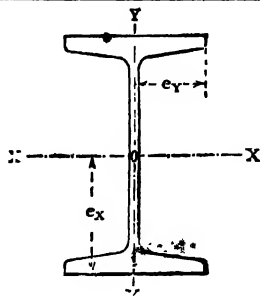
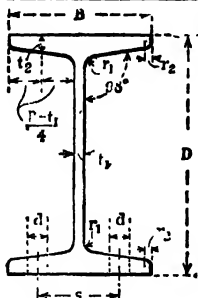
**DIMENSIONS,**  
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**AND**  
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# STEEL JOISTS.



D = Overall depth.  
B = " breadth.  
 $t_1$  = Web thickness.  
 $t_2$  = Flange mean thickness.  
 $r_1$  = Radius of heel fillet.  
 $r_2$  = " " toe "  
s = Spacing of holes. "  
d = Diameter of rivet or bolt.

X-X = Axis of Max. Moment of Inertia and Greatest Radius of Gyration.  
Y-Y = Axis of Min. Moment of Inertia and Least Radius of Gyration.  
O = Centre of Gravity of Section.  
 $e_x$  = Perpendicular distance from X-X to extreme fibre.  
 $e_y$  = " " " Y-Y " " "

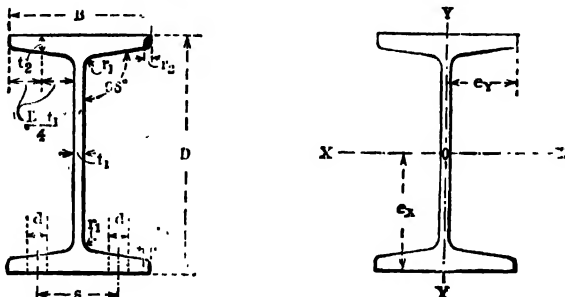
## Dimensions, Position of Centre of Gravity, and Spacing of Holes.

All dimensions are in Inches.

Reference Mark.	Size, D x B inches.	$t_1$	$t_2$	$r_1$	$r_2$	$e_x$	$e_y$	s	d
B.S.B. 30	24 x 7 $\frac{1}{2}$	•600	1•070	•700	•350	12•0	3•75	4•5	$\frac{1}{2}$ or $\frac{3}{4}$
B.S.B. 29	20 x 7 $\frac{1}{2}$	•600	1•010	•700	•350	10•0	3•75	4•5	$\frac{1}{2}$ or $\frac{3}{4}$
B.S.B. 28	18 x 7	•550	•928	•650	•325	9•0	3•5	4•0	$\frac{1}{2}$ or $\frac{3}{4}$
B.S.B. 27	16 x 6	•550	•847	•650	•325	8•0	3•0	3•5	$\frac{1}{2}$
B.S.B. 26	15 x 6	•500	•800	•600	•300	7•5	3•0	3•5	$\frac{1}{2}$
B.S.B. 25	15 x 5	•420	•647	•520	•260	7•5	2•5	2•75	$\frac{1}{2}$
B.S.B. 24	14 x 6a	•500	•878	•600	•300	7•0	3•0	3•5	$\frac{1}{2}$
B.S.B. 23	14 x 6b	•400	•608	•500	•250	7•0	3•0	3•5	$\frac{1}{2}$
B.S.B. 22	12 x 6a	•500	•888	•600	•300	6•0	3•0	3•5	$\frac{1}{2}$
B.S.B. 21	12 x 6b	•400	•717	•500	•250	6•0	3•0	3•5	$\frac{1}{2}$
B.S.B. 20	12 x 5	•350	•550	•450	•225	6•0	2•5	2•75	$\frac{1}{2}$
B.S.B. 19	10 x 8	•600	•970	•700	•350	5•0	4•0	4•75	$\frac{1}{2}$ or $\frac{3}{4}$
B.S.B. 18	10 x 6	•400	•736	•500	•250	5•0	3•0	3•5	$\frac{1}{2}$
B.S.B. 17	10 x 5	•380	•552	•480	•230	5•0	2•5	2•75	$\frac{1}{2}$
B.S.B. 16	9 x 7	•550	•924	•650	•325	4•5	3•5	4•0	$\frac{1}{2}$ or $\frac{3}{4}$

# REDPATH, BROWN & CO., LIMITED.

## STEEL JOISTS.



D = Overall depth.  
 B = " breadth.  
 $t_1$  = Web thickness.  
 $t_2$  = Flange mean thickness.  
 $r_1$  = Radius of heel fillet.  
 $r_2$  = " " toe "  
 s = Spacing of holes.  
 d = Diameter of rivet or bolt.

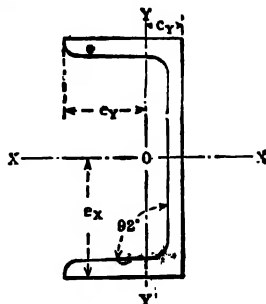
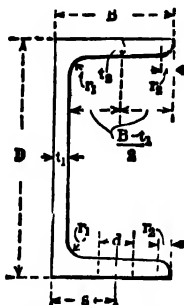
X - X = Axis of Max. Moment of Inertia and Greatest Radius of Gyration.  
 Y - Y = Axis of Min Moment of Inertia and Least Radius of Gyration.  
 O = Centre of Gravity of Section.  
 $e_x$  = Perpendicular distance from X - X to extreme fibre.  
 $e_y$  = " " " Y - Y " " "

### Dimensions, Position of Centre of Gravity, and Spacing of Holes.

All dimensions are in Inches.

Reference Mark.	Size, D x B inches.	$t_1$	$t_2$	$r_1$	$r_2$	$e_x$	$e_y$	s	d
B.S.B. 15	9 x 4	300	460	400	200	4.5	2.0	2.25	$\frac{3}{8}$ or $\frac{1}{2}$
B.S.B. 14	8 x 6	440	597	540	270	4.0	3.0	3.5	$\frac{3}{8}$
B.S.B. 13	8 x 5	350	575	450	225	4.0	2.5	2.75	$\frac{3}{8}$
B.S.B. 12	8 x 4	280	402	380	190	4.0	2.0	2.25	$\frac{3}{8}$ or $\frac{1}{2}$
B.S.B. 11	7 x 4	250	387	350	175	3.5	2.0	2.25	$\frac{3}{8}$ or $\frac{1}{2}$
B.S.B. 10	6 x 5	410	520	510	255	3.0	2.5	2.75	$\frac{3}{8}$
B.S.B. 9	6 x $4\frac{1}{2}$	370	481	470	235	3.0	2.25	2.5	$\frac{3}{8}$
B.S.B. 8	6 x 3	260	348	360	180	3.0	1.5	1.5	$\frac{3}{8}$ or $\frac{1}{2}$
B.S.B. 7	5 x $4\frac{1}{2}$	290	448	390	195	2.5	2.25	2.5	$\frac{3}{8}$
B.S.B. 6	5 x 3	220	370	320	160	2.5	1.5	1.5	$\frac{3}{8}$ or $\frac{1}{2}$
B.S.B. 5	$4\frac{1}{2}$ x $1\frac{1}{2}$	180	325	280	140	2.375	0.875	0.875	$\frac{1}{8}$ or $\frac{1}{4}$
B.S.B. 4	4 x 3	220	336	320	160	2.0	1.5	1.5	$\frac{3}{8}$ or $\frac{1}{2}$
B.S.B. 3	4 x $1\frac{1}{2}$	170	240	270	135	2.0	0.875	0.875	$\frac{1}{8}$ or $\frac{1}{4}$
B.S.B. 2	3 x 3	200	332	300	150	1.5	1.5	1.5	$\frac{3}{8}$ or $\frac{1}{2}$
B.S.B. 1	3 x $1\frac{1}{2}$	180	248	260	130	1.5	0.75	0.75	$\frac{3}{8}$

# STEEL CHANNELS.



D = Overall depth.  
B = " breadth.  
 $t_1$  = Standard web thickness.  
 $t_2$  = " flange mean thickness.  
 $r_1$  = Radius of heel fillet.  
 $r_2$  = " " toe "  
s = Spacing of holes.  
d = Diameter of rivet or bolt.

X-X = Axis of Max. Moment of Inertia and Greatest Radius of Gyration.  
Y-Y = Axis of Min. Moment of Inertia and Least Radius of Gyration.  
O = Centre of Gravity of Section.  
 $e_x$  = Perpendicular distance from X-X to extreme fibre.  
 $e_y$  = " " " Y-Y " " "  
 $c_y$  = " " " Y-Y " web outer surface.

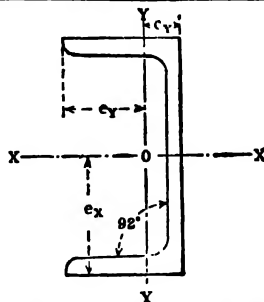
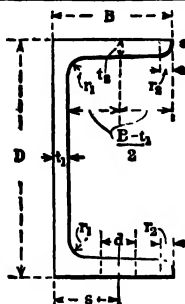
## Dimensions, Position of Centre of Gravity, and Spacing of Holes.

All dimensions are in inches.

Reference Mark.	Size, D x B inches.	$t_1$	$t_2$	$r_1$	$r_2$	$e_x$	$e_y$	$c_y$	s	d
27 N	15 x 4	.625	.630	.630	.440	7.5	3.065	.935	2.25	$\frac{1}{2}$ or $\frac{3}{4}$
26 N	12 x 4	.525	.625	.625	.425	6.0	2.969	1.031	2.25	$\frac{1}{2}$ or $\frac{3}{4}$
25 N	12 x 3 $\frac{1}{2}$	.500	.600	.600	.425	6.0	2.633	.867	2.0	$\frac{1}{2}$
24 N	12 x 3 $\frac{1}{2}$	.375	.500	.500	.350	6.0	2.640	.860	2.0	$\frac{1}{2}$
23 N	11 x 4	.500	.600	.600	.425	5.5	2.937	1.063	2.25	$\frac{1}{2}$ or $\frac{3}{4}$
22 N	11 x 3 $\frac{1}{2}$	.475	.575	.575	.400	5.5	2.604	.896	2.0	$\frac{1}{2}$
21 N	10 x 4	.475	.575	.575	.400	6.0	2.398	1.102	2.25	$\frac{1}{2}$ or $\frac{3}{4}$
20 N	10 x 3 $\frac{1}{2}$	.475	.575	.575	.400	5.0	2.567	.933	2.0	$\frac{1}{2}$
19 N	10 x 3 $\frac{1}{2}$	.375	.500	.500	.350	5.0	2.507	.933	2.0	$\frac{1}{2}$
18 N	9 x 4	.475	.575	.575	.400	4.5	2.849	1.151	2.25	$\frac{1}{2}$ or $\frac{3}{4}$
17 N	9 x 3 $\frac{1}{2}$	.450	.550	.550	.375	4.5	2.529	.971	2.0	$\frac{1}{2}$
16 N	9 x 3 $\frac{1}{2}$	.375	.500	.500	.350	4.5	2.524	.976	2.0	$\frac{1}{2}$
15 N	9 x 3	.375	.437	.437	.350	4.5	2.246	.754	1.75	$\frac{1}{2}$
14 N	8 x 4	.450	.550	.550	.375	4.0	2.790	1.201	2.25	$\frac{1}{2}$ or $\frac{3}{4}$

# REDPATH, BROWN & CO., LIMITED.

## STEEL CHANNELS.



D = Overall depth.  
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 $t_2$  = " flange mean thickness.  
 $r_1$  = Radius of heel fillet.  
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O = Centre of Gravity of Section.  
 $e_x$  = Perpendicular distance from X-X to extreme fibre.  
 $e_y$  = " " " Y-Y " " " "  
 $c_y$  = " " " Y-Y " " web " " surface.

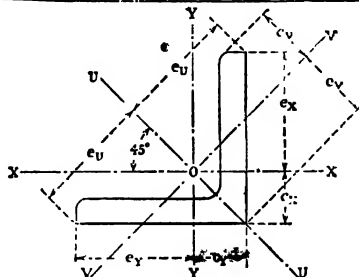
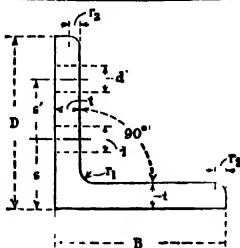
### Dimensions, Position of Centre of Gravity, and Spacing of Holes.

All dimensions are in Inches.

Reference Mark.	Size D x B inches.	$t_1$	$t_2$	$r_1$	$r_2$	$e_x$	$e_y$	$c_y$	s	d
13 N	8 x 3½	·425	·525	·525	·375	4·0	2·489	1·011	2·0	½
12 N	8 x 3	·375	·500	·500	·350	4·0	2·156	·844	1·75	½
11 N	8 x 2½	·312	·437	·437	·300	4·0	1·834	·666	1·375	½
10 N	7 x 3½	·400	·500	·500	·350	3·5	2·439	1·061	2·0	½
9 N	7 x 3	·375	·475	·475	·325	3·5	2·136	·874	1·75	½
8 N	6 x 3½	·375	·475	·475	·325	3·0	2·381	1·119	2·0	½
7 N	6 x 3	·375	·475	·475	·325	3·0	2·072	·928	1·75	½
6 N	6 x 3	·312	·437	·437	·300	3·0	2·062	·988	1·75	½
5 N	6 x 2½	·312	·375	·375	·280	3·0	1·706	·704	1·375	½
4 N	5 x 2½	·312	·375	·375	·280	2·5	1·743	·757	1·375	½
3 N	4 x 2	·250	·375	·375	·280	2·0	1·344	·556	1·125	½
2 N	3½ x 2	·250	·312	·312	·220	1·75	1·355	·645	1·125	½
1 N	3 x 1½	·250	·312	·312	·220	1·5	1·016	·484	0·875	½

# REDPATH, BROWN & CO., LIMITED.

## STEEL EQUAL ANGLES.



D & B = Overall width of lega.  
 t = Thickness of lega.  
 $r_1$  = Radius of heel fillet.  
 $r_2$  = " " toe "  
 s & s' = Spacing of holes.  
 d = Diameter of rivet or bolt.  
 O = Centre of Gravity of Section.

U—U = Axis of Greatest Radius of Gyration as in Part II.  
 V—V = Axis of Least Radius of Gyration as in Part II.  
 X—X, Y—Y = Axes of Moments of Inertia as in Part I.  
 $e_u, e_v, e_x, e_y$  = Perpendicular distances from Axes U—U, V—V, X—X, & Y—Y to extreme fibres.  
 $c_u, c_v, c_x, c_y$  = Perpendicular distances from Axes V—V, X—X, & Y—Y to back lines of Section.

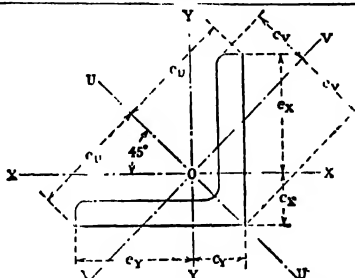
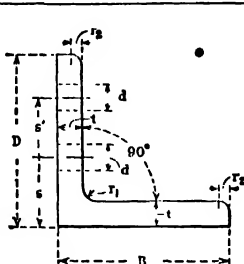
### Dimensions, Position of Centre of Gravity, and Spacing of Holes.

All dimensions are in Inches.

Reference Mark.	Size, D x B x t inches.	$r_1$	$r_2$	$e_u$	$e_v$	$e_x$ $e_y$	$c_v$	$c_x$ $c_y$	s	s'	d
14g Q	0 x 0 x $\frac{3}{8}$	.475	.325	4.24	2.49	4.24	2.15	1.76	2.25	2.25	$\frac{1}{2}$ or $\frac{3}{4}$
14f Q	" x " x $\frac{1}{2}$	"	"	"	2.42	4.29	2.13	1.71	"	"	"
14e Q	" x " x $\frac{1}{2}$	"	"	"	2.35	4.34	2.11	1.66	"	"	"
13g Q	5 x 5 x $\frac{3}{8}$	.425	.300	3.54	2.14	3.49	1.80	1.51	2.0	1.75	$\frac{3}{4}$
13f Q	" x " x $\frac{1}{2}$	"	"	"	2.07	3.54	1.78	1.46	"	"	"
13e Q	" x " x $\frac{1}{2}$	"	"	"	2.00	3.58	1.76	1.42	"	"	"
12g Q	4 $\frac{1}{2}$ x 4 $\frac{1}{2}$ x $\frac{3}{8}$	.400	.275	3.18	1.96	3.11	1.64	1.39	2.5	"	$\frac{3}{4}$
12f Q	" x " x $\frac{1}{2}$	"	"	"	1.90	3.16	1.61	1.34	"	"	"
12e Q	" x " x $\frac{1}{2}$	"	"	"	1.83	3.21	1.59	1.29	"	"	"
11g Q	4 x 4 x $\frac{3}{8}$	.350	.250	2.83	1.79	2.74	1.47	1.26	2.25	"	$\frac{3}{4}$
11f Q	" x " x $\frac{1}{2}$	"	"	"	1.72	2.78	1.44	1.22	"	"	"
11e Q	" x " x $\frac{1}{2}$	"	"	"	1.66	2.83	1.42	1.17	"	"	"
11d Q	" x " x $\frac{1}{2}$	"	"	"	1.59	2.88	1.40	1.12	"	"	"

# REDPATH, BROWN & CO., LIMITED.

## STEEL EQUAL ANGLES.



D & B = Overall width of legs.  
 t = Thickness of legs.  
 $r_1$  = Radius of heel fillet.  
 $r_2$  = " " toe "  
 s &  $s'$  = Spacing of holes.  
 d = Diameter of rivet or bolt.  
 O = Centre of Gravity of Section

U—U = Axis of Greatest Radius of Gyration as in Part II.  
 V—V = Axis of Least Radius of Gyration as in Part II.  
 X—X, Y—Y = Axes of Moments of Inertia as in Part I.  
 $e_U, e_V, e_X, e_Y$  = Perpendicular distances from Axes U—U, V—V, X—X, & Y—Y to extreme fibres.  
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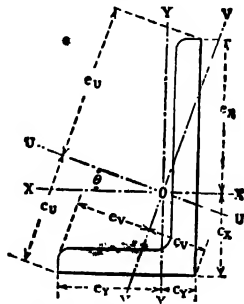
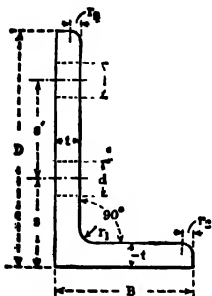
### Dimensions, Position of Centre of Gravity, and Spacing of Holes.

All dimensions are in Inches.

Reference Mark.	Size, D x B x t inches.	$r_1$	$r_2$	$e_U$	$e_V$	$e_X$ $e_Y$	$c_V$	$c_X$ $c_Y$	s	$s'$	d
10f Q	$3\frac{1}{2} \times 3\frac{1}{2} \times \frac{1}{2}$	.325	.225	2.47	1.55	2.41	1.28	1.09	2.0		$\frac{1}{2}$
10e Q	" x " x $\frac{1}{2}$	"	"	"	1.48	2.45	1.25	1.05	"		"
10d Q	" x " x $\frac{1}{2}$	"	"	"	1.41	2.50	1.23	1.00	"		"
9f Q	$3 \times 3 \times \frac{1}{2}$	.300	.200	2.12	1.37	2.03	1.11	0.97	1.75		$\frac{1}{2}$ or $\frac{3}{4}$
9e Q	" x " x $\frac{1}{2}$	"	"	"	1.31	2.08	1.09	0.92	"		"
9d Q	" x " x $\frac{1}{2}$	"	"	"	1.24	2.12	1.06	0.88	"		"
9c Q	" x " x $\frac{1}{2}$	"	"	"	1.21	2.16	1.05	0.85	"		"
9b Q	" x " x $\frac{1}{2}$	"	"	"	1.17	2.17	1.05	0.83	"		"
7e Q	$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{2}$	.275	.200	1.77	1.13	1.70	0.91	0.80	1.375		$\frac{1}{2}$ or $\frac{3}{4}$
7d Q	" x " x $\frac{1}{2}$	"	"	"	1.06	1.75	0.89	0.75	"		"
7c Q	" x " x $\frac{1}{2}$	"	"	"	1.03	1.77	0.87	0.73	"		"
7b Q	" x " x $\frac{1}{2}$	"	"	"	0.99	1.80	0.87	0.70	"		"
6c Q	$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{2}$	.250	.175	1.50	0.94	1.58	0.80	0.67	1.25		$\frac{1}{2}$ or $\frac{3}{4}$
6b Q	" x " x $\frac{1}{2}$	"	"	"	0.91	1.61	0.79	0.64	"		"
5b Q	$2 \times 2 \times \frac{1}{2}$	.250	.175	1.41	0.82	1.42	0.70	0.58	1.125		$\frac{1}{2}$ or $\frac{3}{4}$
5a Q	" x " x $\frac{1}{2}$	"	"	"	0.78	1.45	0.69	0.55	"		"

# REDPATH, BROWN & CO., LIMITED.

## STEEL UNEQUAL ANGLES.



D & B = Overall width of legs.  
t = Thickness of legs.  
r<sub>1</sub> = Radius of heel fillet.  
r<sub>2</sub> = " " toe "  
s & s' = Spacing of holes.  
d = Diameter of rivet or bolt.  
O = Centre of Gravity of Section.

θ = Angle enclosed between

U-U = Axis of Greatest Radius of Gyration as in Part II.

V-V = Axis of Least Radius of Gyration as in Part II.

X-X, Y-Y = Axes of Moments of Inertia as in Part I.

c<sub>u</sub>, c<sub>v</sub>, c<sub>x</sub>, c<sub>y</sub> = Perpendicular distances from axes U-U,

V-V, X-X, and Y-Y to extreme fibres.

c<sub>u</sub>, c<sub>v</sub>, c<sub>x</sub>, c<sub>y</sub> = Perpendicular distances from axes U-U,

V-V, X-X, and Y-Y to back lines of

Section.

Axis X-X and Axis U-U.

" Y-Y " V-V.

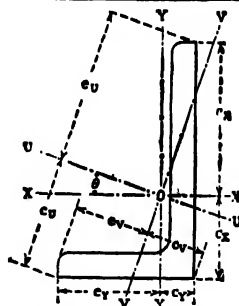
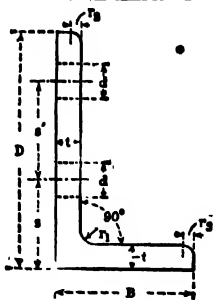
### Dimensions, Position of Centre of Gravity, and Spacing of Holes.

All dimensions are in inches.

Reference Mark.	Size, D × B × t inches.	r <sub>1</sub>	r <sub>2</sub>	c <sub>u</sub>	c <sub>v</sub>	c <sub>x</sub>	c <sub>y</sub>	c <sub>u</sub>	c <sub>v</sub>	c <sub>x</sub>	c <sub>y</sub>	s	s'	d	θ
25g R	7 × 3½ × ½	425	300	4.48	2.03	4.40	2.64	3.16	1.46	2.60	0.86	2.5	3.0	1 or 1½	14
25f R	" × ½	"	"	4.51	2.05	4.45	2.69	3.14	1.42	2.55	0.81	"	"	"	14.5
25e R	" × ¼	"	"	4.55	2.07	4.50	2.74	3.11	1.36	2.50	0.76	"	"	"	14.5
21f R	6 × 4 × ½	425	300	4.06	2.08	3.99	2.98	3.03	1.74	2.01	1.02	2.25	2.25	1 or 1½	23.5
21e R	" × ¼	"	"	4.09	2.03	4.04	3.03	3.01	1.67	1.96	0.97	"	"	"	23.5
20f R	6 × 3½ × ½	400	275	3.90	1.94	3.89	2.63	2.83	1.49	2.11	0.87	2.25	2.25	1 or 1½	18.5
20e R	" × ¼	"	"	3.99	1.92	3.94	2.68	2.85	1.45	2.06	0.82	"	"	"	19.0
20d R	" × ⅛	"	"	4.02	1.96	3.99	2.73	2.79	1.38	2.01	0.77	"	"	"	19.0
63f R	6 × 8 × ½	400	275	3.84	1.76	3.78	2.27	2.70	1.25	2.22	0.78	2.25	2.25	1 or 1½	14.0
63e R	" × ¼	"	"	3.88	1.76	3.88	2.32	2.68	1.23	2.17	0.68	"	"	"	14.5
63d R	" × ⅛	"	"	3.91	1.80	3.88	2.37	2.65	1.14	2.12	0.68	"	"	"	14.5

# REDPATH, BROWN & CO., LIMITED.

## STEEL UNEQUAL ANGLES.



D & B = Overall width of legs.  
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U—U = Axis of Greatest Radius of Gyration as in Part II.

V—V = Axis of Least Radius of Gyration as in Part II.

X—X, Y—Y = Axes of Moments of Inertia as in Part I.

e<sub>u</sub>, e<sub>v</sub>, e<sub>x</sub>, e<sub>y</sub> = Perpendicular distances from axes U—U, V—V, X—X, and Y—Y to extreme fibres.

c<sub>u</sub>, c<sub>v</sub>, c<sub>x</sub>, c<sub>y</sub> = Perpendicular distances from axes U—U, V—V, X—X, and Y—Y to back lines of Section.

θ = Angle enclosed between { Axis X—X and Axis U—U.  
" Y—Y " V—V.

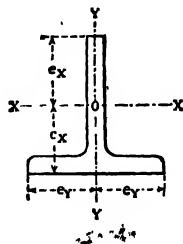
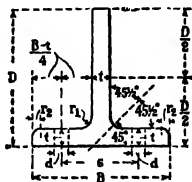
Dimensions, Position of Centre of Gravity, and Spacing of Holes.

All dimensions are in Inches.

Reference Mark.	Size, D × B × t inches.	r <sub>1</sub>	r <sub>2</sub>	e <sub>u</sub>	e <sub>v</sub>	e <sub>x</sub>	e <sub>y</sub>	c <sub>u</sub>	c <sub>v</sub>	c <sub>x</sub>	c <sub>y</sub>	s	s'	d	θ
17f R	5 × 4 × $\frac{5}{16}$	400	275	3.46	1.81	3.40	2.89	2.89	1.79	1.80	1.11	2.0	1.75	$\frac{3}{4}$	32.0
17e R	" × " × $\frac{5}{16}$	"	"	3.48	1.83	3.44	2.94	2.88	1.73	1.56	1.06	"	"	"	32.0
17d R	" × " × $\frac{5}{16}$	"	"	3.49	1.82	3.49	2.99	2.86	1.66	1.51	1.01	"	"	"	32.0
15f R	5 × 3 × $\frac{5}{16}$	350	250	3.30	1.65	3.22	2.21	2.40	1.83	1.78	0.79	2.0	1.75	$\frac{3}{4}$	19.0
15e R	" × " × $\frac{5}{16}$	"	"	3.32	1.65	3.27	2.26	2.38	1.27	1.78	0.74	"	"	"	19.5
15d R	" × " × $\frac{5}{16}$	"	"	3.36	1.67	3.32	2.31	2.37	1.21	1.68	0.69	"	"	"	19.5
11e R	4 × 3 × $\frac{1}{2}$	325	225	2.75	1.45	2.69	2.18	2.19	1.35	1.31	0.82	2.25	"	$\frac{3}{4}$	28.5
11d R	" × " × $\frac{1}{2}$	"	"	2.77	1.45	2.73	2.23	2.18	1.28	1.27	0.77	"	"	"	28.5
7d R	3 × 2½ × $\frac{3}{16}$	275	200	2.09	1.11	2.06	1.80	1.79	1.11	0.94	0.70	1.75	"	$\frac{3}{4}$ or $\frac{1}{2}$	34.0
7e R	" × " × $\frac{3}{16}$	"	"	2.10	1.10	2.08	1.83	1.79	1.07	0.92	0.67	"	"	"	34.0

# REDPATH, BROWN & CO., LIMITED.

## STEEL TEES.



B=Overall width of table.  
D= " depth over stalk.  
t=Mean Thickness (table and stalk).  
 $r_1$ =Radius of heel fillet.  
 $r_2$ = " " toe "  
s=Spacing of holes.  
d=Diameter of rivet or bolt.

X-X, Y-Y=Axes of Max. and Min. Moments of Inertia and Greatest and Least Radii of Gyration.

O=Centre of Gravity of Section.

$e_x, e_y$ =Perpendicular distances from Axes X-X and Y-Y to extreme fibres.

$c_x$ =Perpendicular distance from X-X to table outer surface.

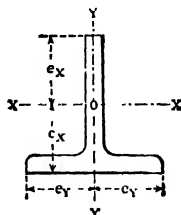
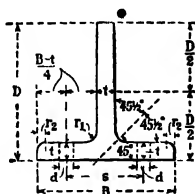
### Dimensions, Position of Centre of Gravity, and Spacing of Holes.

All Dimensions are in Inches.

Reference Mark.	Size, B x D x t inches.	$r_1$	$r_2$	$e_x$	$e_y$	$c_x$	s	d
21s W	6 x 4 x $\frac{1}{2}$	.425	.300	3.03	3.0	0.97	3.5	$\frac{1}{2}$ or $\frac{3}{4}$
20s W	6 x 3 x $\frac{1}{2}$	.400	.275	2.32	3.0	0.68	3.5	$\frac{1}{2}$ or $\frac{3}{4}$
20d W	" x $\frac{1}{2}$	"	"	2.37	3.0	0.63	"	"
19s W	5 x 4 x $\frac{1}{2}$	.400	.275	2.95	2.5	1.05	2.75	$\frac{3}{4}$
19d W	" x $\frac{1}{2}$	"	"	3.00	2.5	1.00	"	"
17s W	5 x 3 x $\frac{1}{2}$	.350	.250	2.26	2.5	0.74	2.75	$\frac{3}{4}$
17d W	" x $\frac{1}{2}$	"	"	2.31	2.5	0.69	"	"
16s W	4 x 5 x $\frac{1}{2}$	.400	.275	3.47	2.0	1.53	2.25	$\frac{3}{4}$
10d W	" x $\frac{1}{2}$	"	"	3.53	2.0	1.47	"	"

# REDPATH, BROWN & CO., LIMITED.

## STEEL TEES.



$B$  = Overall width of table.  
 $D$  = " depth over stalk.  
 $t$  = Mean Thickness (table and stalk).  
 $r_1$  = Radius of heel fillet.  
 $r_2$  = " " toe.  
 $s$  = Spacing of holes.  
 $d$  = Diameter of rivet or bolt.

$X-X, Y-Y$  = Areas of Max. and Min. Moments of Inertia and Greatest and Least Radii of Gyration.

$O$  = Centre of Gravity of Section.

$e_x, e_y$  = Perpendicular distances from Axes  $X-X$  and  $Y-Y$  to extreme fibres.

$c_x$  = Perpendicular distance from  $X-X$  to table outer surface.

### Dimensions, Position of Centre of Gravity, and Spacing of Holes.

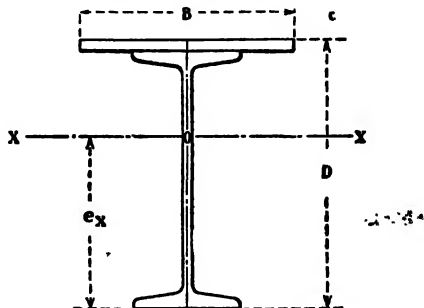
All Dimensions are in Inches.

Reference Mark.	Size, $B \times D \times t$ inches.	$r_1$	$r_2$	$e_x$	$e_y$	$c_x$	$s$	$d$
15c W	$4 \times 4 \times \frac{1}{2}$	.350	.250	2.34	2.0	1.16	2.25	$\frac{3}{4}$
15d W	" $\times \frac{3}{8}$	"	"	2.89	2.0	1.11	"	"
14c W	$4 \times 3 \times \frac{1}{2}$	.325	.225	2.18	2.0	0.82	2.25	$\frac{3}{4}$
14d W	" $\times \frac{3}{8}$	"	"	2.23	2.0	0.77	"	"
13c W	$3\frac{1}{2} \times 3\frac{1}{2} \times \frac{1}{2}$	.325	.225	2.46	1.75	1.04	2.0	$\frac{3}{4}$
13d W	" $\times \frac{3}{8}$	"	"	2.51	1.75	0.99	"	"
11c W	$3 \times 3 \times \frac{1}{2}$	.300	.200	2.08	1.50	0.92	1.5	$\frac{3}{4}$ or $\frac{1}{2}$
11d W	" $\times \frac{3}{8}$	"	"	2.13	1.50	0.87	"	"
8d W	$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{3}{8}$	.275	.200	1.75	1.25	0.75	1.25	$\frac{1}{2}$
8b W	" $\times \frac{1}{4}$	"	"	1.80	1.25	0.70	"	"

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## COMPOUND GIRDERS.

(Part I., pages 68-69).



B = Overall width of flange plate.  
D = Overall depth.  
X—X = Axis of Max. Mom. of Inertia. (See Part I.).  
O = Centre of Gravity of Section.  
 $e_x$  = Perpendicular distance from Axis X—X to extreme fibre, in inches.

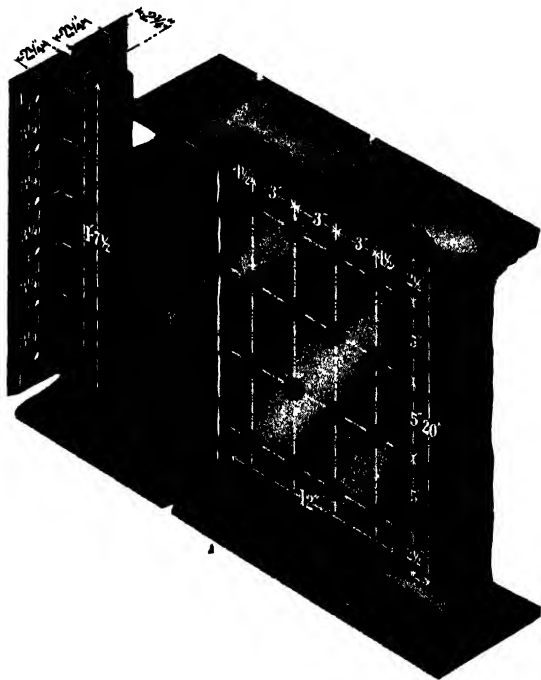
### Position of Centre of Gravity.

Reference Mark.	Size, D x B inches.	$e_x$	Reference Mark.	Size, D x B inches.	$e_x$
29 D	16 $\frac{1}{2}$ x 12	10.42	15 D	12 $\frac{1}{2}$ x 12	8.00
28 D	16 $\frac{1}{2}$ x "	10.04	14 D	12 $\frac{1}{2}$ x "	7.60
27 D	15 $\frac{1}{2}$ x "	9.86	13 D	12 $\frac{1}{2}$ x 10	8.52
26 D	15 $\frac{1}{2}$ x "	9.50	12 D	12 $\frac{1}{2}$ x "	8.17
25 D	16 $\frac{1}{2}$ x 10	10.12	11 D	12 $\frac{1}{2}$ x "	7.70
24 D	15 $\frac{1}{2}$ x "	9.78	10 D	10 $\frac{1}{2}$ x 12	7.01
23 D	15 $\frac{1}{2}$ x "	9.20	9 D	10 $\frac{1}{2}$ x "	6.72
22 D	14 $\frac{1}{2}$ x 12	9.26	8 D	10 $\frac{1}{2}$ x 10	7.20
21 D	14 $\frac{1}{2}$ x "	9.00	7 D	10 $\frac{1}{2}$ x "	6.89
20 D	14 $\frac{1}{2}$ x "	9.62	6 D	10 $\frac{1}{2}$ x "	6.55
19 D	14 $\frac{1}{2}$ x "	9.23	5 D	8 $\frac{1}{2}$ x 12	5.82
18 D	12 $\frac{1}{2}$ x "	8.02	4 D	8 $\frac{1}{2}$ x "	5.56
17 D	12 $\frac{1}{2}$ x "	7.70	3 D	8 $\frac{1}{2}$ x "	5.86
16 D	12 $\frac{1}{2}$ x "	8.32	2 D	8 $\frac{1}{2}$ x 10	5.60
			1 D	8 $\frac{1}{2}$ x "	5.29

REDPATH, BROWN & CO., LIMITED.

**END ANGLES AND FISHPLATES.**

**STANDARD DETAILS.**



**STEEL JOIST, 24" x 7½" x 100 LBS.**

**ANGLE CLEATS, 4" x 4" x ½" x 7½". FISHPLATES, 20" x 12" x ½".**

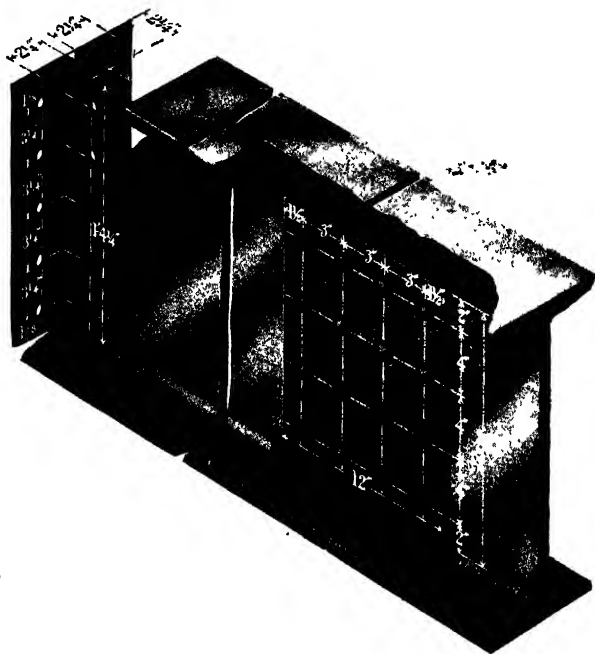
**MINIMUM SPAN, 16 FEET. HOLES, ½" DIAMETER.**

**MAXIMUM LOAD, 34 TONS. BOLTS OR RIVETS, ¾" DIAMETER.**

REDPATH, BROWN & CO., LIMITED.

**END ANGLES AND FISHPLATES.**

STANDARD DETAILS.



**STEEL JOIST, 20" x 7½" x 89 LBS.**

**ANGLE CLEATS, 4" x 4½" x 1¼" FISHPLATES, 16" x 12" x ½".**

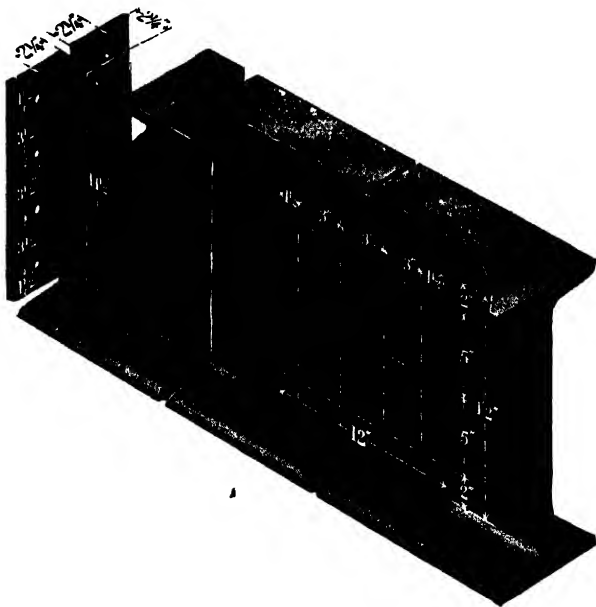
**MINIMUM SPAN, 15 FEET. HOLES, ⅝" DIAMETER.**

**MAXIMUM LOAD, 29 TONS. BOLTS OR RIVETS, ⅞" DIAMETER.**

REDPATH, BROWN & CO., LIMITED.

**END ANGLES AND FISHPLATES.**

**STANDARD DETAILS.**



**STEEL JOIST, 18" x 7" x 75 LBS.**

**ANGLE CLEATS, 4" x 4" x 1/2" x 1 1/2"      FISHPLATES, 14" x 12" x 1/2".**

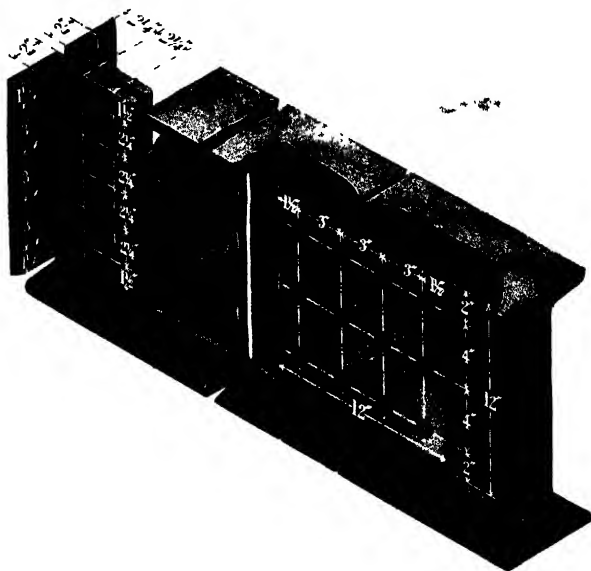
**MINIMUM SPAN, 15 FEET.      HOLES, 1 1/8" DIAMETER.**

**MAXIMUM LOAD, 21 TONS.      BOLTS OR RIVETS, 3/4" DIAMETER.**

REDPATH, BROWN & CO., LIMITED.

**END ANGLES AND FISHPLATES.**

**STANDARD DETAILS.**



**STEEL JOIST, 16" x 6" x 62 LBS.**

**ANGLE CLEATS, 6" x 3 1/2" x 1/2" x 1' 0"** **FISHPLATES, 12" x 12" x 1/2".**

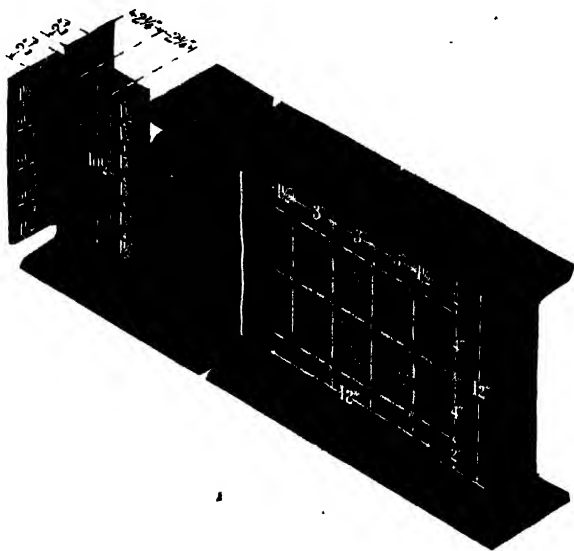
**MINIMUM SPAN, 11 FEET.** **HOLES, 3/16" DIAMETER.**

**MAXIMUM LOAD, 22 TONS.** **BOLTS OR RIVETS, 3/4" DIAMETER.**

REDPATH, BROWN & CO., LIMITED.

## END ANGLES AND FISHPLATES.

STANDARD DETAILS.



## STEEL JOISTS.

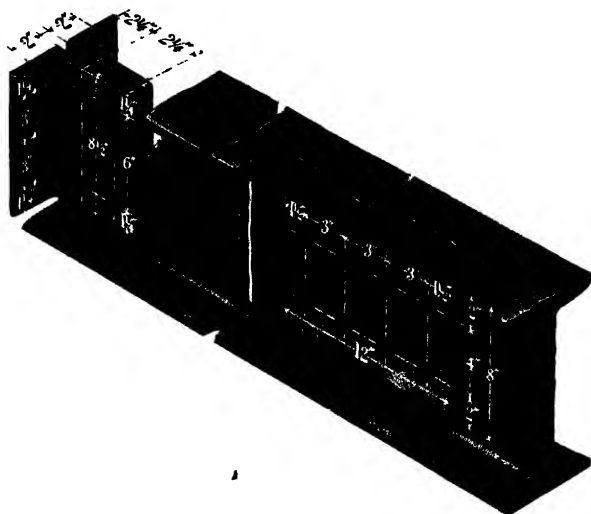
STEEL JOISTS	MIN. SPAN.	MAX. LOAD.
15" x 6" x 59 LBS.	10 FEET.	20 TONS.
15" x 5" x 42 "	9 "	17 "
ANGLE CLEATS. 6" x 3½" x ½" x 10½"	HOLES, ⅝" DIAMETER.	
FISHPLATES. 12" x 12" x ½"	BOLTS OR RIVETS, ¾" DIAMETER.	



REDPATH, BROWN & CO., LIMITED.

## END ANGLES AND FISHPLATES.

### STANDARD DETAILS.



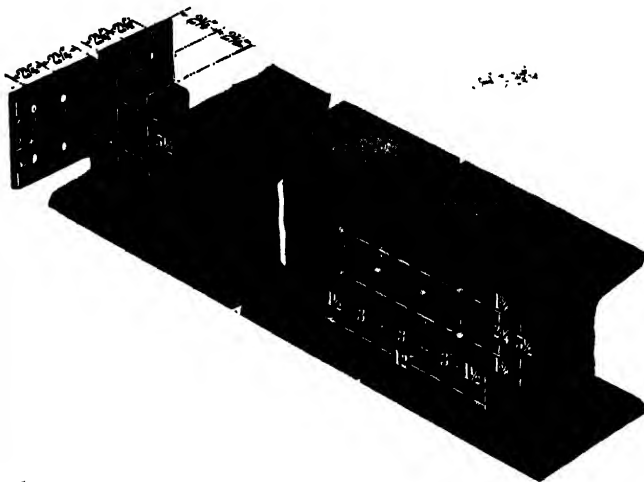
## STEEL-JOISTS.

STEEL JOISTS.	MIN SPAN.	MAX. LOAD.
12"x6" x 54 LBS.	10 FEET.	16 TONS.
12"x6" x 44 "	10 "	13 "
12"x5" x 32 "	8 "	11 "
ANGLE CLEATS. 6"x3½"x½"x8½"	HOLES. ⅜" DIAMETER.	
FISHPLATES. 8" x 12" x ½".	BOLTS OR RIVETS. ¾" DIAMETER.	

REDPATH, BROWN & CO., LIMITED.

**END ANGLES AND FISHPLATES.**

**STANDARD DETAILS.**



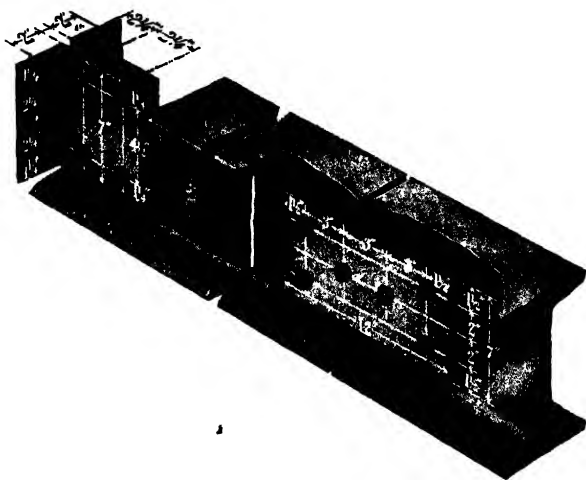
**STEEL JOISTS**

STEEL JOISTS.	MIN SPAN.	MAX. LOAD.
10" x 8" x 70 LBS.	8 FEET.	23 TONS.
9" x 7" x 58 "	6 "	21 "
ANGLE CLEATS. 6" x 6" x 1/2" x 5 1/2".	HOLES. 1 5/16" DIAMETER.	
FISHPLATES. 5 1/2" x 12" x 1/2".	BOLTS OR RIVETS. 3/8" DIAMETER.	

REDPATH, BROWN & CO., LIMITED.

**END ANGLES AND FISHPLATES.**

**STANDARD DETAILS.**

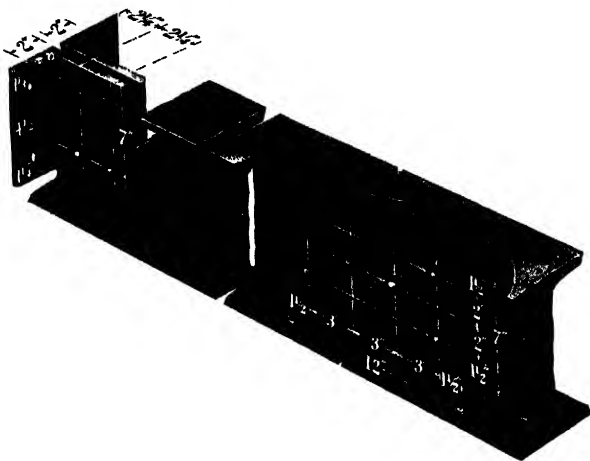


**STEEL JOIST.** 10' x 6" x 42 LBS.  
**ANGLE CLEATS.** 6"x3½"x½"x7" **FISHPLATES.** 7"x12'x½".  
**MINIMUM SPAN.** 8 FEET. **HOLES.** ⅝" DIAMETER.  
**MAXIMUM LOAD.** 13 TONS. **BOLTS OR RIVETS.** ¾" DIAMETER.

REDPATH, BROWN & CO., LIMITED.

## END ANGLES AND FISHPLATES.

STANDARD DETAILS.



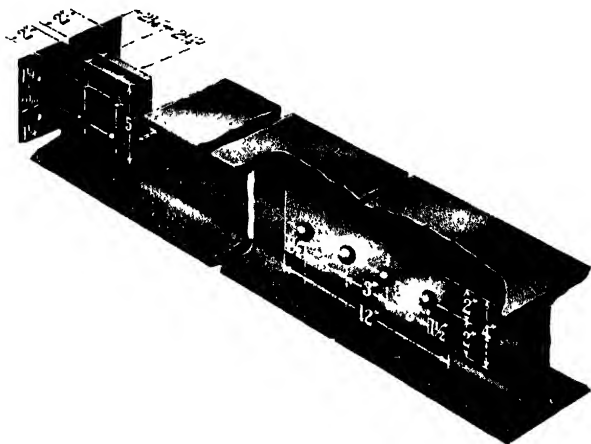
## STEEL JOISTS.

STEEL JOISTS.	MIN. SPAN.	MAX. LOAD.
10" x 5" x 30 LBS.	6 FEET.	12 TONS.
9" x 4" x 21 "	5 "	10 "
ANGLE CLEATS. 6" x 3½" x ½" x 7"	HOLES. ⅜" DIAMETER.	
FISHPLATES. 7" x 12" x ½".	BOLTS OR RIVETS. ¾" DIAMETER.	

REDPATH, BROWN & CO., LIMITED.

## END ANGLES AND FISHPLATES.

STANDARD DETAILS.



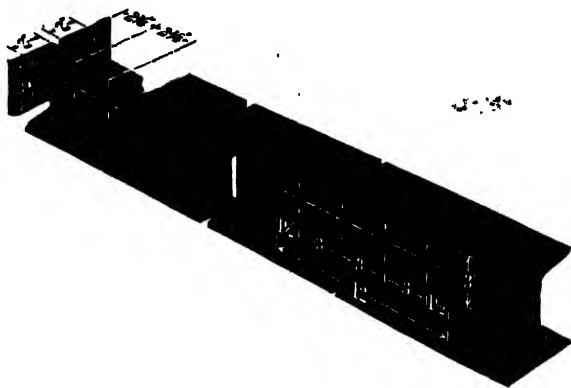
## STEEL JOISTS.

STEEL JOISTS.	MIN SPAN.	MAX. LOAD.
8" x 6" x 35 LBS.	5 FEET.	14 TONS.
8" x 5" x 28 "	5 "	11 "
8" x 4" x 18 "	4 "	9 "
7" x 4" x 16 "	4 "	8 "
ANGLE CLEATS. 6" x 3 1/2" x 1/2" x 5".	HOLES, 13/16" DIAMETER.	
FISHPLATES. 4" x 12" x 1/2".	BOLTS OR RIVETS, 3/4" DIAMETER.	

REDPATH, BROWN & CO., LIMITED.

## END ANGLES AND FISHPLATES.

STANDARD DETAILS.\*



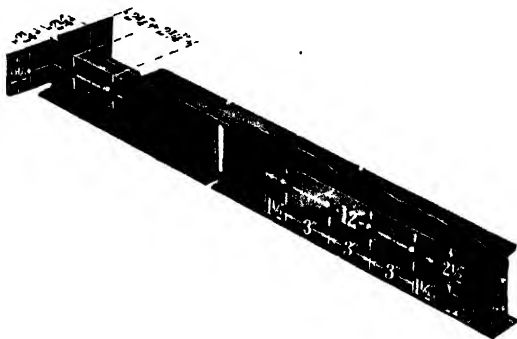
## STEEL JOISTS.

STEEL JOISTS.	MIN SPAN.	MAX. LOAD.
6" x 5" x 25 LBS.	6 FEET.	6 TONS.
6" x 4½" x 20 "	5 "	6 "
6" x 3" x 12 "	4 "	4 "
5" x 4½" x 18 "	4 "	6 "
5" x 3" x 11 "	3 "	5 "
ANGLE CLEATS. 6" x 3½" x ½" x 3"	HOLES. ⅜" DIAMETER.	
FISHPLATES. 3" x 12" x ½".	BOLTS OR RIVETS. ¾" DIAMETER.	

REDPATH, BROWN & CO., LIMITED.

**END ANGLES AND FISHPLATES.**

**STANDARD DETAILS.**



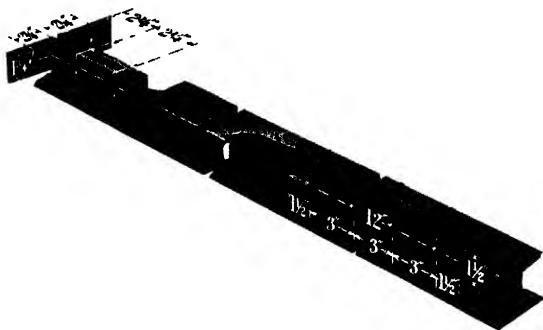
**STEEL JOISTS.**

STEEL JOISTS.	MIN. SPAN.	MAX. LOAD.
$4\frac{3}{4} \times 1\frac{3}{4} \times 6\frac{1}{2}$ LBS.	2 FEET.	4 TONS.
$4 \times 3 \times 9\frac{1}{2}$ ..	2 ..	4 ..
$4 \times 1\frac{3}{4} \times 5$ ..	2 ..	3 ..
ANGLE CLEATS. $6 \times 3\frac{1}{2} \times 1\frac{1}{2} \times 2\frac{1}{2}$ .	HOLES. $\frac{1}{16}$ DIAMETER.	
FISHPLATES. $2\frac{1}{2} \times 12 \times \frac{1}{2}$ .	BOLTS OR RIVETS. $\frac{5}{16}$ DIAMETER.	

REDPATH, BROWN & CO., LIMITED.

## END ANGLES AND FISHPLATES.

STANDARD DETAILS.



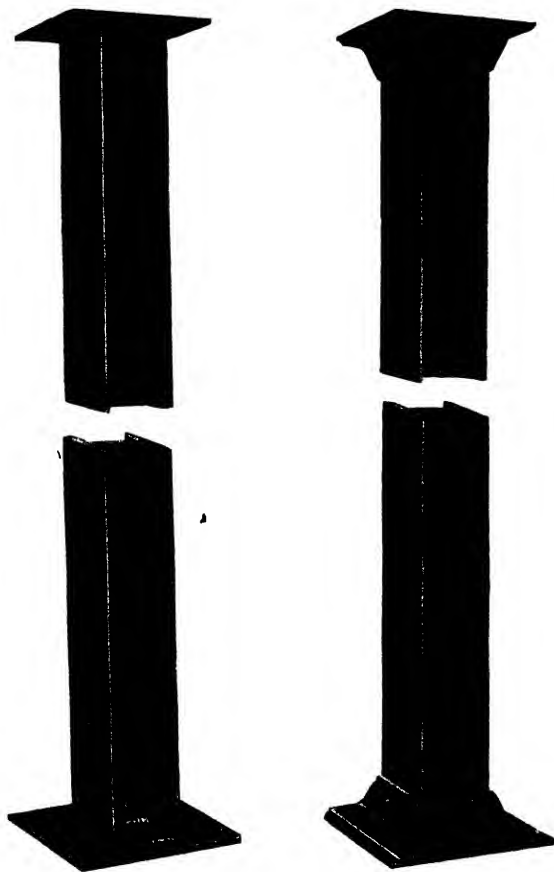
## STEEL JOISTS.

STEEL JOISTS.	MIN. SPAN.	MAX. LOAD.
3" x 3" x 8½ LBS.	2 FEET.	4 TONS.
3" x 1½" x 4 "	1 "	3 "
ANGLE GLEATS. 6" x 3½" x 1½" x 1¼"	HOLES. ¼" DIAMETER.	
FISHPLATES. 1½" x 12" x ½"	BOLTS OR RIVETS. ⅝" DIAMETER.	

REDPATH, BROWN & CO., LIMITED.

**STANCHION CAPS AND BASES.**

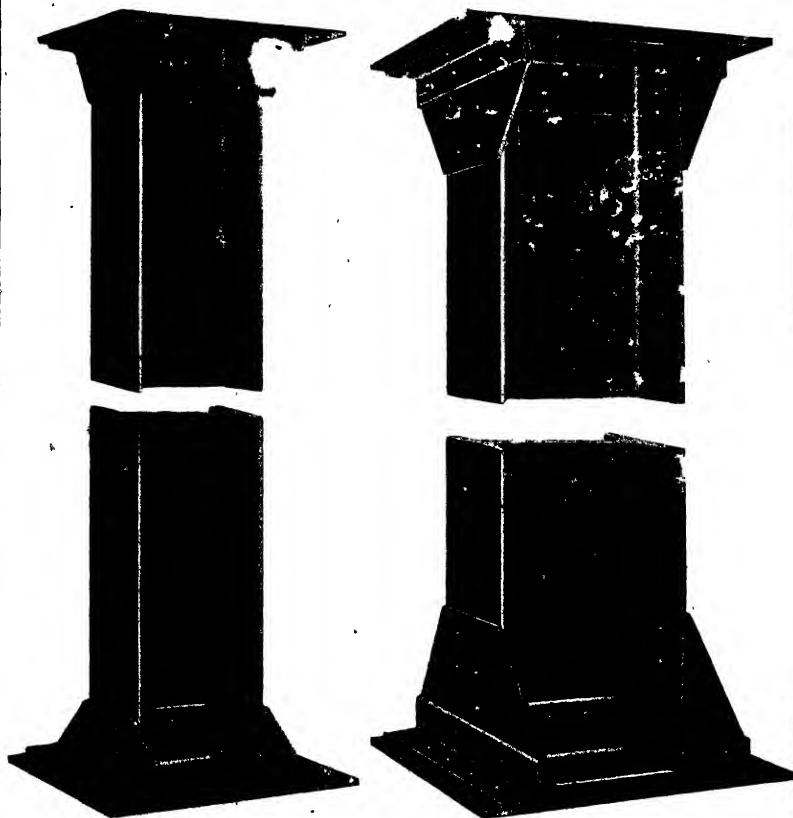
• TYPICAL DETAILS.



REDPATH, BROWN & CO., LIMITED.

**STANCHION CAPS AND BASES.**

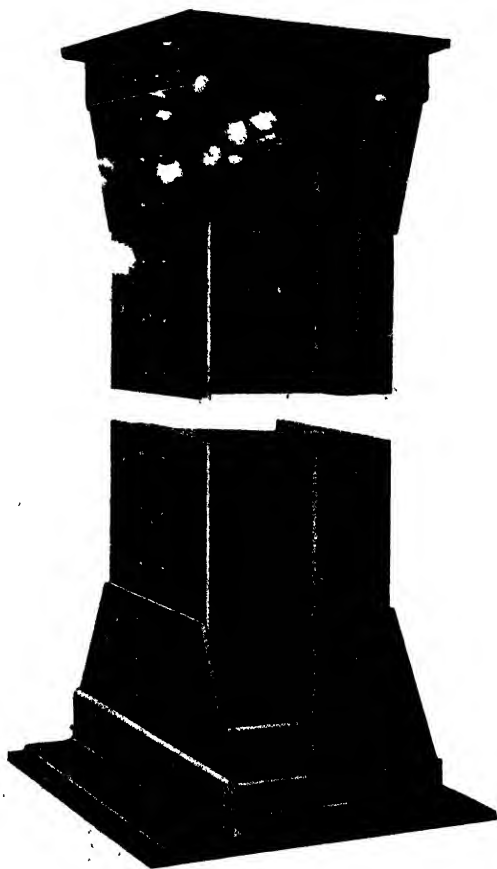
TYPICAL DETAILS. :



REDPATH, BROWN & CO., LIMITED.

**STANCHION CAPS AND BASES.**

• TYPICAL DETAILS.



REDPATH, BROWN & CO., LIMITED.

**STANCHION CAPS AND BASES.**

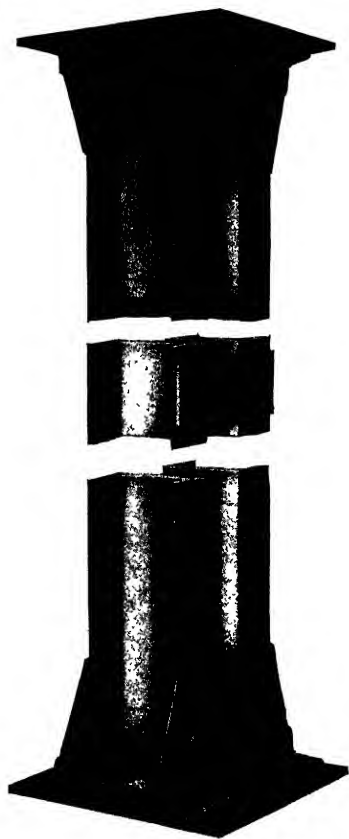
**TYPICAL DETAILS.**



REDPATH, BROWN & CO., LIMITED.

**STANCHION CAPS AND BASES.**

• TYPICAL DETAILS.



REDPATH, BROWN & CO., LIMITED.

**STANCHION CAPS AND BASES.**

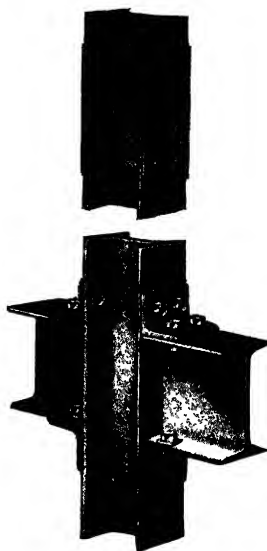
TYPICAL DETAILS.



REDPATH, BROWN & CO., LIMITED.

**SPLICES AND CONNECTIONS.**

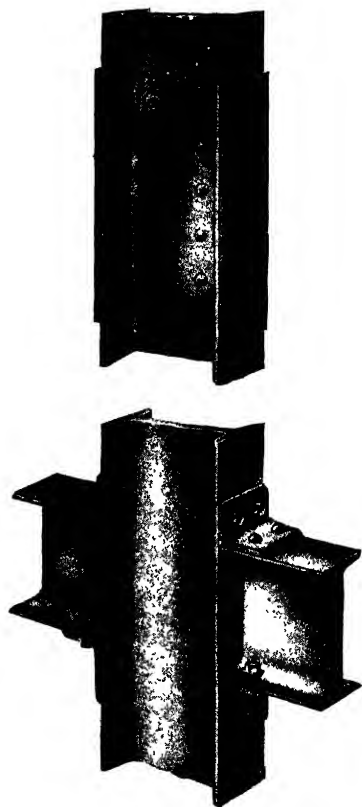
• TYPICAL DETAILS.



REDPATH, BROWN & CO., LIMITED.

**SPLICES AND CONNECTIONS.**

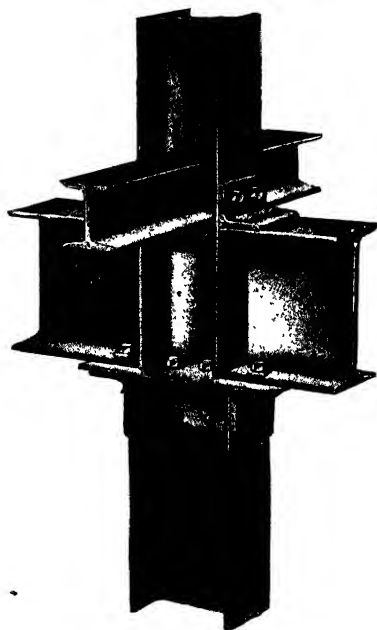
TYPICAL DETAILS. ✓



REDPATH, BROWN & CO., LIMITED.

**SPLICES AND CONNECTIONS.**

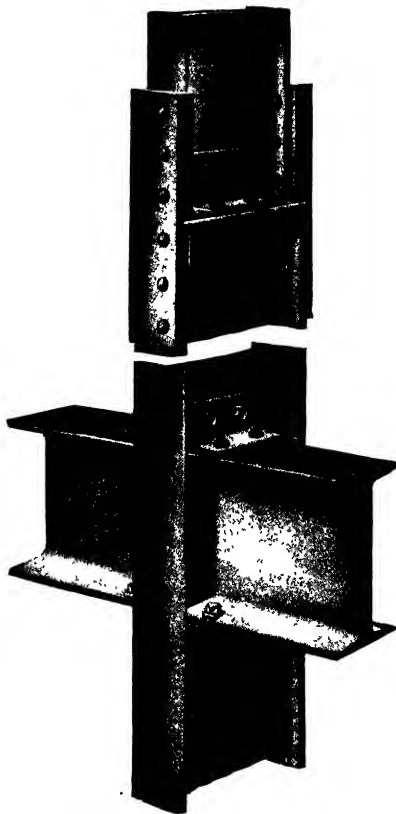
• TYPICAL DETAILS.



REDPATH, BROWN & CO., LIMITED.

**SPLICES AND CONNECTIONS.**

TYPICAL DETAILS. \*

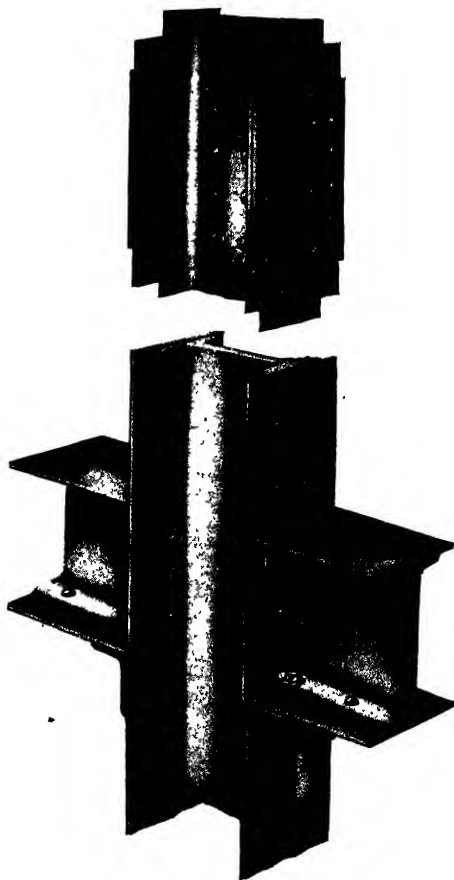


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**SPLICES AND CONNECTIONS.**

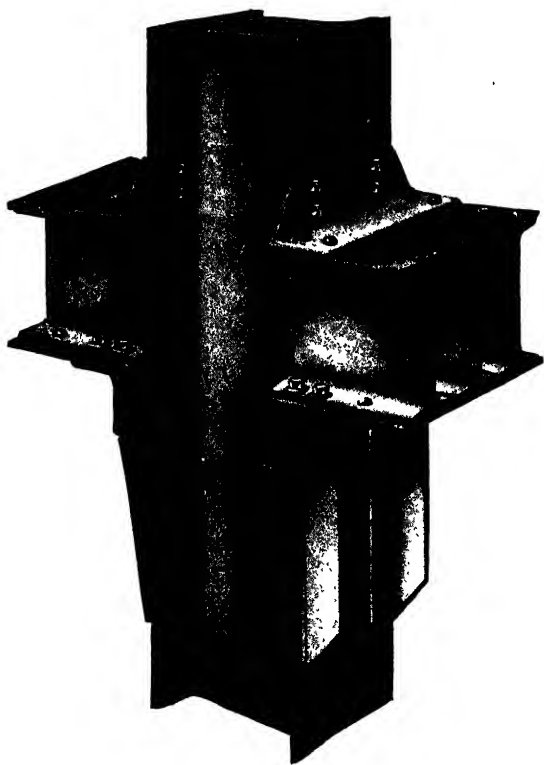
• TYPICAL DETAILS.



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**CONNECTION.**

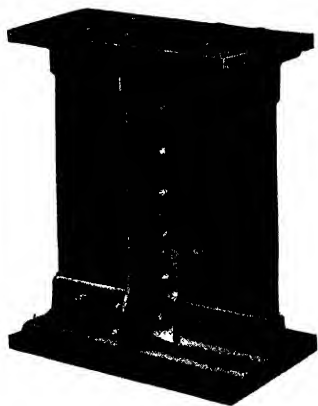
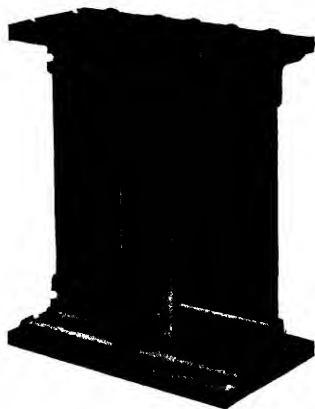
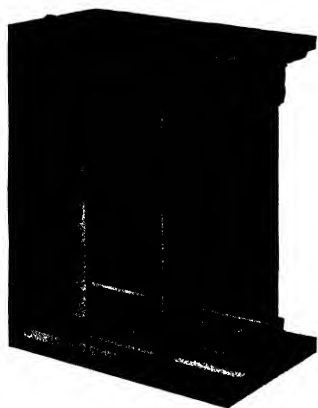
TYPICAL DETAIL. '



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**STIFFENERS.**

•TYPICAL DETAILS.



ROOF DETAILS.



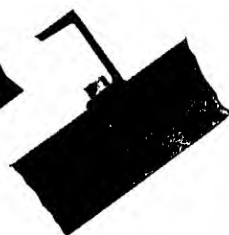
APEX.



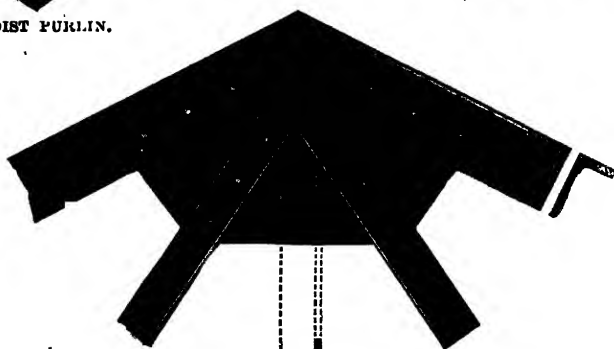
JOIST PURLIN.



ANGLE PURLIN.



CHANNEL PURLIN.



APEX.

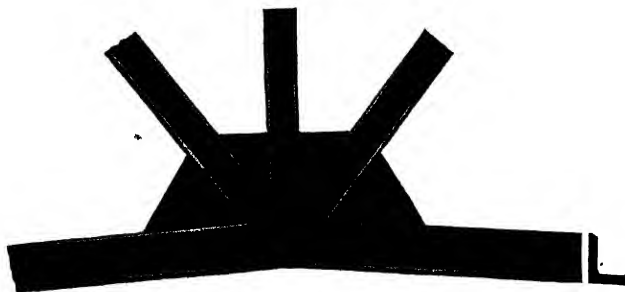
ROOF DETAILS.



RAFTER STRUT AND TIES.



MAIN TIES, STRUT AND DIAGONAL TIE.



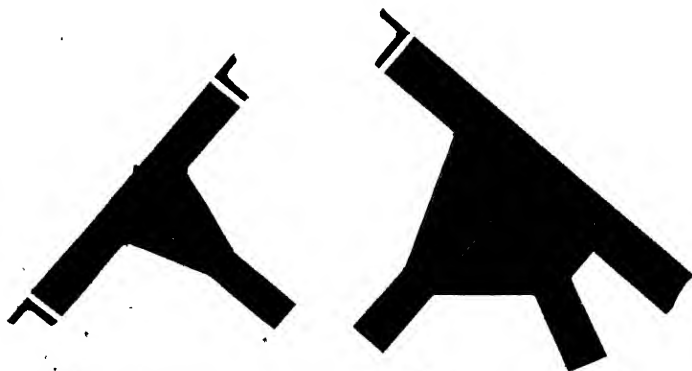
MAIN TIES, STRUTS AND KING TIE.

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ROOF DETAILS.



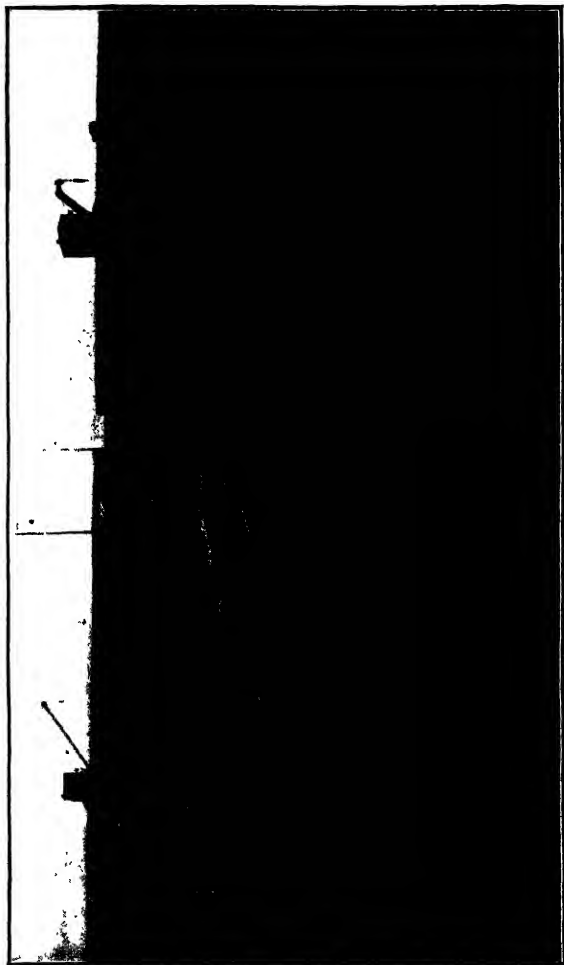
SHOES.



RAFTER AND STRUT.

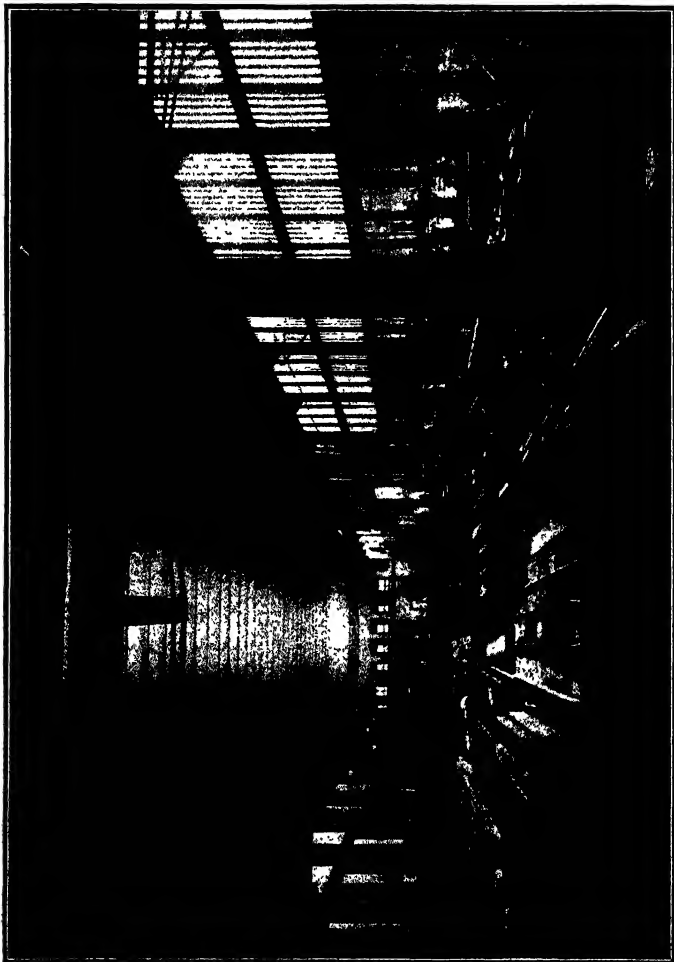
RAFTER STRUT AND TIE.

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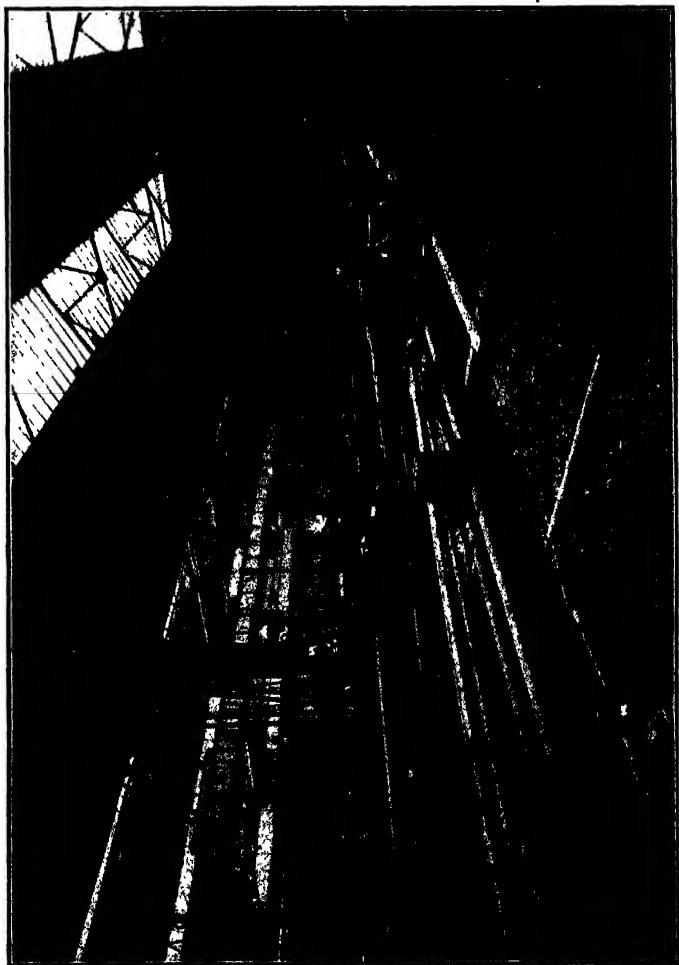
A Portion of One of the Stockyards.

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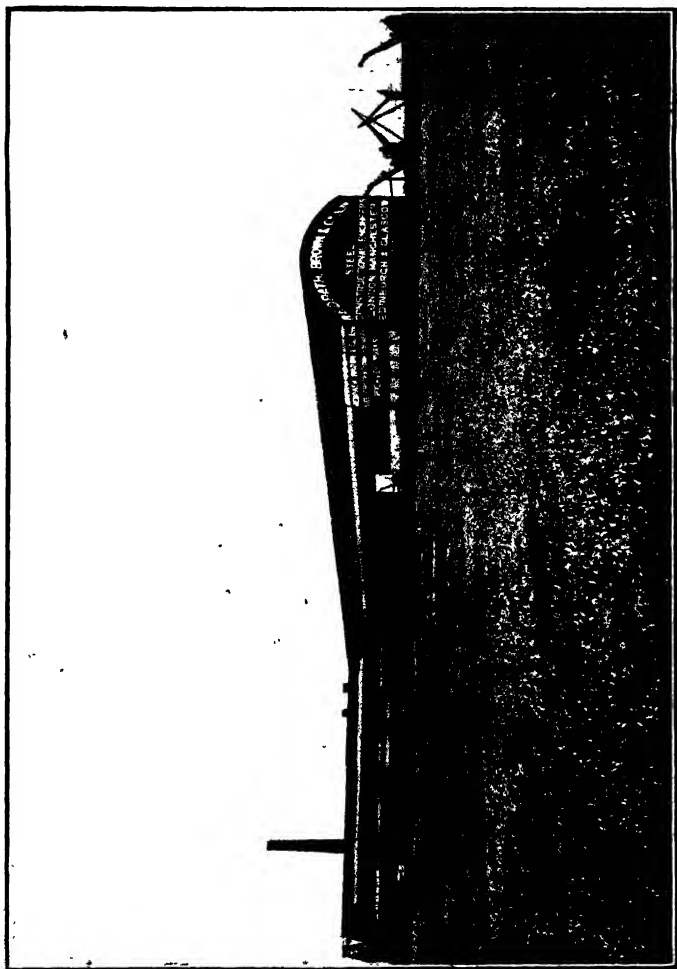
An Interior of One of the Works.

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An Interior of One of the Works.

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General View of One of the Works.

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REDPATH, BROWN & CO., LIMITED.

PART VI

(LONDON)

STANCHIONS.

SAFE LOADS

in accordance with the  
L.C.C. (General Powers) Act, 1908

AND

PROPERTIES,

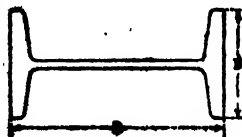
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# REDPATH, BROWN & CO., LIMITED.



## STANCHIONS.

### Steel Joists.

Safe Concentric Loads, in Tons.  
Ends Fixed.

Reference Mark.	Size, D x B inches.	HEIGHTS IN FEET.											
		8	9	10	11	12	13	14	16	18	20	22	24
30 J	24 x 7½	144	138	132	126	120	115	109	97.5	85.8	74.1		
29 J	20 x 7½	129	124	119	114	109	104	99.0	88.9	78.7	68.6		
28 J	18 x 7	106	102	97.8	93.3	88.7	84.1	79.6	70.5	61.4			
27 J	16 x 6	82.6	78.1	73.6	69.1	64.6	60.1	55.6	46.7				
26 J	15 x 6	80.1	76.0	71.9	67.8	63.7	59.6	55.5	47.4				
25 J	15 x 5	50.1	46.3	42.5	38.8	35.0	31.2						
24 J	14 x 6a	77.8	73.9	70.0	66.1	62.2	58.3	54.4	46.6				
23 J	14 x 6b	62.2	59.0	55.8	52.6	49.3	46.1	42.9	36.5				
22 J	12 x 6a	74.6	71.0	67.5	63.9	60.3	56.7	53.2	46.0				
21 J	12 x 6b	60.4	57.4	54.4	51.4	48.4	45.4	42.4	36.7				
20 J	12 x 5	38.9	36.2	33.4	30.6	27.8	25.1						
19 J	10 x 8	107	104	100	97.3	94.0	90.7	87.4	80.8	74.1	67.5	60.9	54.3
18 J	10 x 6	58.5	55.8	53.1	50.3	47.6	44.9	42.2	36.7	31.3			
17 J	10 x 5	37.2	34.7	32.2	29.6	27.1	24.6	22.1					
16 J	9 x 7	36.0	32.9	29.8	26.7	23.6	20.5	17.4	15.4	9.4	8.7		

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160.

Safe loads are in accordance with the working stresses prescribed by the London County Council (General Powers) Act, 1900 for stanchions of mild steel having "both ends fixed."

For other conditions and formulae, see notes commencing page 122 L.

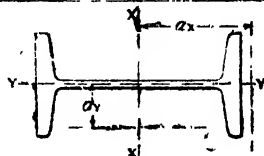
For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## STANCHIONS.

### Steel Joists.

#### Dimensions and Properties



Size, D x B inches.	Weight per foot in lbs.	Area in square inches.	Standard Thicknesses		Radii of Gyration.		Eccentricity Coefficients.			
			Web	Flange	Axis Y-Y	Axis X-X	Web.	Flange	Axis Y-Y	Axis X-X
24 x 7½	100	29.392	600	1070	1.51	9.50	1.50	2.60	1 + 1.65a <sub>v</sub>	1 + 0.13a <sub>x</sub>
20 x 7½	89	26.161	600	1010	1.54	7.90	1.47	2.57	1 + 1.57a <sub>v</sub>	1 + 0.16a <sub>x</sub>
18 x 7	75	22.006	550	928	1.45	7.22	1.46	2.56	1 + 1.66a <sub>v</sub>	1 + 0.17a <sub>x</sub>
16 x 6	62	18.227	550	847	1.22	6.31	1.58	2.61	1 + 2.02a <sub>v</sub>	1 + 0.20a <sub>x</sub>
15 x 6	59	17.346	500	880	1.27	6.02	1.46	2.55	1 + 1.85a <sub>v</sub>	1 + 0.21a <sub>x</sub>
15 x 5	42	12.351	420	647	0.98	5.80	1.55	2.62	1 + 2.59a <sub>v</sub>	1 + 0.22a <sub>x</sub>
14 x 6a	57	16.709	500	873	1.29	5.64	1.45	2.54	1 + 1.80a <sub>v</sub>	1 + 0.22a <sub>x</sub>
14 x 6b	46	13.533	400	698	1.26	5.70	1.38	2.51	1 + 1.88a <sub>v</sub>	1 + 0.22a <sub>x</sub>
12 x 6a	54	15.879	500	883	1.33	4.86	1.42	2.52	1 + 1.69a <sub>v</sub>	1 + 0.26a <sub>x</sub>
12 x 6b	44	12.946	400	717	1.31	4.93	1.35	2.48	1 + 1.75a <sub>v</sub>	1 + 0.25a <sub>x</sub>
12 x 5	32	9.408	350	550	1.02	4.83	1.42	2.51	1 + 2.41a <sub>v</sub>	1 + 0.26a <sub>x</sub>
10 x 8	70	20.582	600	970	1.86	4.00	1.35	2.49	1 + 1.15a <sub>v</sub>	1 + 0.30a <sub>x</sub>
10 x 6	42	12.358	400	736	1.36	4.14	1.33	2.46	1 + 1.62a <sub>v</sub>	1 + 0.29a <sub>x</sub>
10 x 5	30	8.820	360	552	1.05	4.06	1.41	2.52	1 + 2.26a <sub>v</sub>	1 + 0.30a <sub>x</sub>
9 x 7	58	17.064	550	924	1.64	3.67	1.36	2.51	1 + 1.29a <sub>v</sub>	1 + 0.34a <sub>x</sub>

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

Each weight per foot is for the shaft only. Weight of base, &c., to be added.

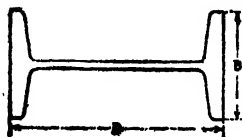
Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

W = actual eccentric load; K = relative eccentricity coefficient; Wc = equivalent concentric value; Wm = WexK.

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for a<sub>v</sub> and a<sub>x</sub> respectively.

For full explanations of tables, see notes commencing page 192 L.

# REDPATH, BROWN & CO., LIMITED.



## STANCHIONS.

### Steel Joists.

Safe Concentric Loads, in Tons.  
Ends Fixed.

Reference Mark.	Size, D × R inches	HEIGHTS IN FEET.												
		3	4	5	6	7	8	9	10	11	12	14	16	
15 J	9 × 4	33.4	31.1	28.9	26.6	24.4	22.1	19.9	17.6	15.4				
14 J	8 × 6	59.8	57.5	55.2	52.8	50.5	48.1	45.8	43.5	41.1	38.8	34.1	29.4	
13 J	8 × 5	46.9	44.7	42.4	40.2	38.0	35.8	33.6	31.3	29.1	26.9	22.5		
12 J	8 × 4	28.6	26.6	24.7	22.8	20.8	18.9	17.0	15.0	13.1				
11 J	7 × 4	25.6	23.9	22.3	20.6	18.9	17.3	15.6	14.0	12.3				
10 J	6 × 5	41.8	39.8	37.8	35.9	33.9	31.9	29.9	27.9	25.9	24.0	20.0		
9 J	6 × 4½	32.7	30.8	29.0	27.1	25.3	23.5	21.6	19.8	17.9	16.1			
8 J	6 × 3	17.7	16.0	14.3	12.6	10.9	9.1							
7 J	5 × 4½	29.7	28.2	26.7	25.1	23.6	22.1	20.5	19.0	17.5	15.9			
6 J	5 × 3	16.7	15.2	13.8	12.3	10.9	9.4	8.0						
4 J	4 × 3	14.4	13.2	11.9	10.7	9.4	8.2	7.0						
2 J	3 × 3	13.0	12.0	10.9	9.9	8.8	7.8	6.7						
1 J	3 × 1½	4.3	3.3											

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160.

Safe loads are in accordance with the working stresses prescribed by the London County Council (General Powers) Act, 1909, for stanchions of mild steel having "both ends fixed."

For other conditions and formulae, see notes commencing page 192 L.

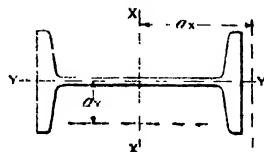
For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## STANCHIONS.

### Steel Joists.

#### Dimensions and Properties.



Size, D x B inches.	Weight per foot in lbs.	Area in square inches.	Standard Thicknesses.		Radii of Gyration.		Eccentricity Coefficients.			
			Web	Flange.	Axis Y-Y	Axis X-X	Web.	Flange.	Axis Y-Y	Axis X-X
9 x 4	21	6.178	.300	.460	0.82	3.62	1.44	2.54	$1 + 2.95a_y$	$1 + 0.34a_x$
8 x 6	35	10.293	.440	.597	1.32	3.28	1.38	2.49	$1 + 1.72a_y$	$1 + 0.37a_x$
8 x 5	28	8.241	.350	.575	1.11	3.29	1.35	2.48	$1 + 2.01a_y$	$1 + 0.37a_x$
8 x 4	18	5.297	.280	.402	0.82	3.24	1.42	2.52	$1 + 2.97a_y$	$1 + 0.38a_x$
7 x 4	16	4.709	.250	.387	0.85	2.88	1.35	2.47	$1 + 2.76a_y$	$1 + 0.42a_x$
6 x 5	25	7.354	.410	.520	1.11	2.43	1.42	2.52	$1 + 2.02a_y$	$1 + 0.51a_x$
6 x 4½	20	5.882	.370	.431	0.96	2.42	1.45	2.53	$1 + 2.45a_y$	$1 + 0.51a_x$
6 x 3	12	3.527	.260	.348	0.61	2.39	1.52	2.57	$1 + 3.96a_y$	$1 + 0.53a_x$
5 x 4½	18	5.290	.290	.448	1.03	2.07	1.31	2.46	$1 + 2.11a_y$	$1 + 0.58a_x$
5 x 3	11	3.238	.220	.376	0.67	2.05	1.37	2.49	$1 + 3.32a_y$	$1 + 0.60a_x$
4 x 3	9½	2.795	.220	.336	0.67	1.64	1.36	2.49	$1 + 3.28a_y$	$1 + 0.74a_x$
3 x 3	8½	2.501	.200	.332	0.71	1.23	1.30	2.49	$1 + 2.98a_y$	$1 + 0.99a_x$
3 x 1½	4	1.176	.160	.248	0.32	1.18	1.57	2.60	$1 + 7.10a_y$	$1 + 1.07a_x$

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

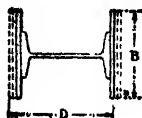
Each weight per foot is for the shaft only. Weight of base, &c., to be added.

Least radii of gyration and relative eccentricity coefficients are printed in prominent type.  
We=actual eccentric load; K=relative eccentricity coefficient; Wc=equivalent concentric value;  $Wc = We \times K$ .

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for  $a_y$  and  $a_x$  respectively.

For full explanations of tables, see notes commencing page 192 L.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND STANCHIONS.

Safe Concentric Loads, in Tons.  
Ends Fixed.

Reference Mark.	Size, D x B inches.	HEIGHTS IN FEET.											
		10	11	12	13	14	16	18	20	24	28	32	36
278 K	27 x 14	398	391	384	378	371	358	345	332	305	279	252	226
276 K	26½ x "	357	351	344	338	332	320	308	295	271	246	222	197
274 K	26 x "	316	310	304	299	293	281	270	259	236	213	190	168
273 K	25½ x "	295	289	284	278	273	262	251	240	218	196	174	152
272 K	25¼ x "	274	269	263	258	253	242	232	221	200	179	157	136
271 K	25½ x "	253	248	243	238	232	222	212	202	181	161	140	120
258 K	23 x 14	381	375	369	362	356	344	332	319	294	270	245	220
256 K	22½ x "	340	334	329	323	317	306	295	283	260	238	215	192
254 K	22 x "	299	294	289	283	278	268	257	247	226	205	184	163
253 K	21½ x "	279	274	268	263	258	248	238	228	208	188	168	148
252 K	21¼ x "	258	253	248	243	239	229	219	210	190	171	152	132
251 K	21½ x "	237	232	228	223	218	209	200	191	172	153	135	116
238 K	21 x 12	316	310	304	298	292	280	268	256	231	207	183	159
236 K	20½ x "	282	276	271	265	260	249	237	226	204	181	159	137
234 K	20 x "	248	243	237	232	227	217	207	196	176	155	135	
233 K	19½ x "	230	225	221	216	211	201	191	181	162	142	122	
232 K	19¼ x "	213	208	204	199	194	185	175	166	147	128	110	
231 K	19½ x "	195	191	186	182	177	168	159	150	133	115	97.0	

Rivets ¾-in. diam. at 6-in. pitch.

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160.

Safe loads are in accordance with the working stresses prescribed by the London County Council (General Powers) Act, 1900, for stanchions of mild steel having "both ends fixed."

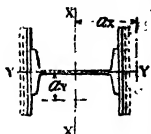
For other conditions and formulae, see notes commencing page 192 L.

For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED

## COMPOUND STANCHIONS.

Composition and Properties.



Composed of		Weight per foot in lbs.	Area in square inches.	Radii of Gyration.		Eccentricity Coefficients.			
One Steel Joist.	Plates, each flange to form.			Axis Y-Y	Axis X-X	Web.	Flange.	Axis Y-Y	Axis X-X
24 x 7½	14 x 1½	246½	71.4	3.24	11.53	1.20	2.37	1 + 0.67a <sub>v</sub>	1 + 0.10a <sub>x</sub>
"	" x 1½	228	64.4	3.15	11.31	1.21	2.37	1 + 0.71a <sub>v</sub>	1 + 0.11a <sub>x</sub>
"	" x 1	199	57.4	3.03	11.07	1.23	2.38	1 + 0.76a <sub>v</sub>	1 + 0.11a <sub>x</sub>
"	" x ¾	187	53.9	2.94	10.94	1.24	2.39	1 + 0.81a <sub>v</sub>	1 + 0.11a <sub>x</sub>
"	" x ¾	175	50.4	2.85	10.79	1.26	2.40	1 + 0.86a <sub>v</sub>	1 + 0.11a <sub>x</sub>
"	" x ¾	163½	46.9	2.74	10.64	1.28	2.41	1 + 0.93a <sub>v</sub>	1 + 0.11a <sub>x</sub>
20 x 7½	14 x 1½	235½	68.1	3.31	9.79	1.19	2.38	1 + 0.64a <sub>v</sub>	1 + 0.12a <sub>x</sub>
"	" x 1½	212	61.1	3.22	9.59	1.20	2.38	1 + 0.68a <sub>v</sub>	1 + 0.12a <sub>x</sub>
"	" x 1	188	51.1	3.10	9.37	1.22	2.38	1 + 0.73a <sub>v</sub>	1 + 0.13a <sub>x</sub>
"	" x ¾	179½	50.6	3.02	9.25	1.23	2.38	1 + 0.77a <sub>v</sub>	1 + 0.13a <sub>x</sub>
"	" x ¾	164	47.1	2.93	9.13	1.25	2.39	1 + 0.82a <sub>v</sub>	1 + 0.13a <sub>x</sub>
"	" x ¾	152½	43.6	2.82	8.99	1.27	2.40	1 + 0.88a <sub>v</sub>	1 + 0.13a <sub>x</sub>
18 x 7	12 x 1½	201	58.0	2.87	8.88	1.20	2.40	1 + 0.73a <sub>v</sub>	1 + 0.14a <sub>x</sub>
"	" x 1½	181	52.0	2.79	8.69	1.21	2.39	1 + 0.77a <sub>v</sub>	1 + 0.14a <sub>x</sub>
"	" x 1	160½	46.0	2.69	8.48	1.23	2.39	1 + 0.83a <sub>v</sub>	1 + 0.14a <sub>x</sub>
"	" x ¾	150	43.0	2.63	8.37	1.24	2.39	1 + 0.87a <sub>v</sub>	1 + 0.14a <sub>x</sub>
"	" x ¾	140	40.0	2.56	8.26	1.25	2.40	1 + 0.92a <sub>v</sub>	1 + 0.15a <sub>x</sub>
"	" x ¾	130	37.0	2.47	8.13	1.27	2.40	1 + 0.98a <sub>v</sub>	1 + 0.15a <sub>x</sub>

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

Each weight per foot is for the riveted shaft only. Weight of base, &c., to be added.

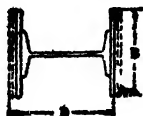
Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

We=actual eccentric load;  $\alpha$ =relative eccentricity coefficient;  $W_c$ =equivalent concentric value;  $W_c = W \times K$ .

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for  $a_v$  and  $a_x$  respectively.

For full explanations of tables, see notes commencing page 192 L.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND STANCHIONS.

Safe Concentric Loads, in Tons.  
Ends Fixed.

Reference Mark	Slenderness D x K inches	HEIGHTS IN FEET													
		10	11	12	13	14	16	18	20	24	28	32	36		
215 K	19 x 12	296	290	285	279	274	263	251	240	218	195	173	151		
216 K	18 1/2 x	262	257	252	247	241	231	221	211	190	170	150	129		
214 K	18 x	228	223	218	214	209	200	190	181	163	144	126	107		
213 K	17 1/2 x	210	206	202	197	193	184	175	166	149	131	113			
212 K	17 1/4 x	193	189	185	180	176	168	159	151	134	117	101			
211 K	17 1/8 x	176	172	168	164	160	152	144	136	120	104	88 1			
210 K	17 x "	158	154	150	146	143	135	127	120	104	89 7				
198 L	18 1/2 x 12	242	236	231	225	220	209	200	190	171	154	137	120		
196 K	17 1/2 x	258	253	248	243	238	228	218	208	188	168	149	129		
194 K	17 x	223	219	214	210	205	196	185	179	161	143	125	107		
193 K	16 1/2 x	206	202	198	193	189	181	172	164	147	130	113	96 0		
191 K	16 1/4 x	189	185	181	177	173	165	157	149	132	116	100			
190 K	16 1/8 x	172	168	164	160	156	149	141	133	115	103	87 8			
190 L	16 x	154	150	147	143	139	132	125	118	105	90 0	74 4			
178 K	17 x 12	289	284	278	273	267	257	246	235	214	192	171	149		
176 K	16 1/2 x	255	250	245	240	235	225	216	206	187	167	148	128		
174 K	16 x	221	216	212	207	203	194	185	177	159	142	124	106		
173 K	15 1/2 x	203	199	195	191	187	178	170	162	145	128	112	95 6		
172 K	15 x	186	182	178	174	170	162	155	147	131	115	100			
171 K	15 1/4 x	169	165	161	158	154	146	139	132	117	102	87 2			
170 K	15 x "	151	148	144	141	137	130	123	116	102	88 1	73 9			

Rivets 3/4 in diam at 6 in pitch

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160.

Safe loads are in accordance with the working stresses prescribed by the London County Council (General Powers) Act, 1909, for stanchions of mild steel having "both ends fixed"

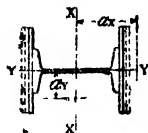
For other conditions and formulae, see notes commencing page 192 L.

For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND STANCHIONS.

### Composition and Properties.



Composed of		Weight per foot in lbs.	Area in square inches.	Radii of Gyration		Eccentricity Coefficients.			
One steel Joist.	Plates, each flange to form			Axle Y-Y	Axle X-X	Web.	Flange.	Axle Y-Y	Axle X-X
16 x 6	12 x 1 1/4	187	54.2	2.91	8.92	1.20	2.41	1 + 0.71a <sub>v</sub>	1 + 0.15a <sub>x</sub>
"	" x 1 1/4	166 1/2	48.2	2.83	7.83	1.21	2.40	1 + 0.75a <sub>v</sub>	1 + 0.15a <sub>x</sub>
"	" x 1	145	42.2	2.73	7.63	1.22	2.39	1 + 0.81a <sub>v</sub>	1 + 0.16a <sub>x</sub>
"	" x 7/8	136	39.2	2.66	7.54	1.23	2.39	1 + 0.85a <sub>v</sub>	1 + 0.16a <sub>x</sub>
"	" x 3/4	125 1/2	36.2	2.59	7.41	1.25	2.40	1 + 0.90a <sub>v</sub>	1 + 0.16a <sub>x</sub>
"	" x 5/8	115 1/2	33.2	2.49	7.28	1.27	2.40	1 + 0.97a <sub>v</sub>	1 + 0.16a <sub>x</sub>
"	" x 1/2	105 1/2	30.2	2.38	7.14	1.29	2.42	1 + 1.06a <sub>v</sub>	1 + 0.17a <sub>x</sub>
15 x 6	12 x 1 1/4	184	53.3	2.93	7.60	1.18	2.40	1 + 0.70a <sub>v</sub>	1 + 0.16a <sub>x</sub>
"	" x 1 1/4	163 1/2	47.3	2.86	7.43	1.18	2.39	1 + 0.73a <sub>v</sub>	1 + 0.16a <sub>x</sub>
"	" x 1	143	41.3	2.76	7.24	1.20	2.38	1 + 0.79a <sub>v</sub>	1 + 0.16a <sub>x</sub>
"	" x 7/8	133	38.3	2.70	7.13	1.21	2.38	1 + 0.82a <sub>v</sub>	1 + 0.17a <sub>x</sub>
"	" x 3/4	122 1/2	35.3	2.63	7.05	1.22	2.38	1 + 0.87a <sub>v</sub>	1 + 0.17a <sub>x</sub>
"	" x 5/8	112 1/2	32.3	2.55	6.91	1.24	2.38	1 + 0.93a <sub>v</sub>	1 + 0.17a <sub>x</sub>
"	" x 1/2	102 1/2	29.3	2.42	6.78	1.26	2.39	1 + 1.02a <sub>v</sub>	1 + 0.18a <sub>x</sub>
14 x 6	12 x 1 1/4	182	52.7	2.95	7.17	1.17	2.41	1 + 0.69a <sub>v</sub>	1 + 0.17a <sub>x</sub>
"	" x 1 1/4	161 1/2	46.7	2.88	6.98	1.18	2.40	1 + 0.73a <sub>v</sub>	1 + 0.17a <sub>x</sub>
"	" x 1	141	40.7	2.78	6.80	1.20	2.39	1 + 0.78a <sub>v</sub>	1 + 0.18a <sub>x</sub>
"	" x 7/8	131	37.7	2.72	6.70	1.20	2.38	1 + 0.81a <sub>v</sub>	1 + 0.18a <sub>x</sub>
"	" x 3/4	120 1/2	34.7	2.65	6.59	1.22	2.38	1 + 0.86a <sub>v</sub>	1 + 0.18a <sub>x</sub>
"	" x 5/8	110 1/2	31.7	2.56	6.48	1.23	2.38	1 + 0.92a <sub>v</sub>	1 + 0.18a <sub>x</sub>
"	" x 1/2	100 1/2	28.7	2.44	6.36	1.25	2.39	1 + 1.01a <sub>v</sub>	1 + 0.19a <sub>x</sub>

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2 1/2 per cent. over this must be allowed. See page 7.

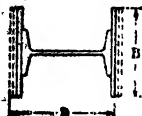
Each weight per foot is for the riveted shaft only. Weight of base, &c., to be added.

Least radii of gyration and relative eccentricity coefficients are printed in prominent type. We=actual eccentric load; K=relative eccentricity coefficient; We=equivalent concentric value; We=We x K.

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for a<sub>v</sub> and a<sub>x</sub> respectively.

For full explanations of tables, see notes commencing page 192 L.

# REDPATH, BROWN & CO., LIMITED



## COMPOUND STANCHIONS.

Safe Concentric Loads, in Tons.  
Ends Fixed.

Reference Mark.	Size, D x B inches.	HEIGHTS IN FEET.												
		10	11	12	13	14	16	18	20	24	28	32	36	
158 K	17 x 12	272	267	263	258	253	243	233	223	204	184	164	145	
156 K	16½ x "	238	234	230	225	221	212	203	194	177	159	141	124	
154 K	16 x "	204	200	196	193	189	181	173	165	149	134	118	102	
153 K	15½ x "	187	183	180	176	172	165	158	150	136	121	106	91.9	
152 K	15¼ x "	170	167	163	160	156	149	142	136	122	108	94.7	80.9	
151 K	15¼ x "	153	150	146	143	140	133	127	121	108	95.3	82.4		
150 K	15 x "	135	132	129	126	123	117	111	105	93.8	81.7	69.7		
138 K	15 x 12	284	279	274	269	264	253	243	232	211	190	169	149	
136 K	14½ x "	250	246	241	236	231	222	213	203	184	165	146	127	
134 K	14 x "	216	212	208	204	199	191	182	174	157	140	123	106	
133 K	13¾ x "	199	195	191	187	183	175	167	159	143	127	111	95.1	
132 K	13½ x "	182	178	174	171	167	159	152	144	129	114	99.0		
131 K	13¼ x "	165	161	157	154	150	143	136	129	115	100	86.5		
130 K	13 x "	147	144	140	137	134	127	120	113	100	87.0	73.5		
118 K	15 x 12	269	265	260	255	250	241	231	221	202	183	163	144	
116 K	14½ x "	235	231	227	223	218	210	201	192	175	158	140	123	
114 K	14 x "	201	198	194	190	186	178	171	163	148	133	117	102	
113 K	13¾ x "	184	181	177	174	170	163	156	149	134	120	106	91.7	
112 K	13½ x "	167	164	161	157	154	147	140	134	120	107	94.1	80.7	
111 K	13¼ x "	150	147	144	141	138	131	125	119	106	94.3	81.9		
110 K	13 x "	133	130	127	124	121	115	110	104	92.6	81.0	69.4		

Rivets ¾-in. diam. at 6-in. pitch.

Rivets ½-in. diam. at 6-in. pitch.

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160.

Safe loads are in accordance with the working stresses prescribed by the London County Council (General Powers) Act, 1900, for stanchions of mild steel having "both ends fixed."

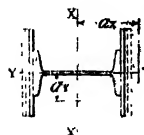
For other conditions and formulæ, see notes commencing page 192 L.

For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND STANCHIONS.

### Composition and Properties.



Composed of		Weight per foot in lbs.	Area in square inches	Radii of Gyration.		Eccentricity Coefficients.			
One Steel Joist.	Plates, each flange to form.			Axis Y-Y	Axis X-X	Web.	Flange	Axis Y-Y	Axis X-X
11 x 6b	12 x 1 1/2	171	49.5	3.02	7.26	1.13	2.37	1 + 0.66a <sub>v</sub>	1 + 0.16a <sub>x</sub>
"	" x 1 1/4	150 1/2	43.5	2.96	7.09	1.14	2.36	1 + 0.69a <sub>v</sub>	1 + 0.17a <sub>x</sub>
"	" x 1	130	37.5	2.87	6.91	1.15	2.34	1 + 0.73a <sub>v</sub>	1 + 0.17a <sub>x</sub>
"	" x 7/8	120	34.5	2.81	6.84	1.15	2.34	1 + 0.76a <sub>v</sub>	1 + 0.17a <sub>x</sub>
"	" x 3/4	109 1/2	31.5	2.74	6.71	1.16	2.34	1 + 0.80a <sub>v</sub>	1 + 0.17a <sub>x</sub>
"	" x 5/8	99 1/2	28.5	2.66	6.60	1.17	2.34	1 + 0.85a <sub>v</sub>	1 + 0.18a <sub>x</sub>
"	" x 1/2	89 1/2	25.5	2.54	6.48	1.19	2.34	1 + 0.93a <sub>v</sub>	1 + 0.18a <sub>x</sub>
12 x 6a	12 x 1 1/2	179	51.8	2.98	6.24	1.17	2.45	1 + 0.68a <sub>v</sub>	1 + 0.19a <sub>x</sub>
"	" x 1 1/4	158 1/2	45.8	2.91	6.08	1.18	2.42	1 + 0.71a <sub>v</sub>	1 + 0.20a <sub>x</sub>
"	" x 1	138	39.8	2.81	5.91	1.19	2.41	1 + 0.76a <sub>v</sub>	1 + 0.20a <sub>x</sub>
"	" x 7/8	128	36.8	2.75	5.81	1.20	2.40	1 + 0.79a <sub>v</sub>	1 + 0.21a <sub>x</sub>
"	" x 3/4	117 1/2	33.8	2.68	5.72	1.21	2.40	1 + 0.83a <sub>v</sub>	1 + 0.21a <sub>x</sub>
"	" x 5/8	107 1/2	30.8	2.59	5.61	1.22	2.39	1 + 0.89a <sub>v</sub>	1 + 0.21a <sub>x</sub>
"	" x 1/2	97 1/2	27.8	2.48	5.50	1.24	2.40	1 + 0.97a <sub>v</sub>	1 + 0.22a <sub>x</sub>
12 x 6b	12 x 1 1/2	169	48.9	3.04	6.33	1.13	2.41	1 + 0.65a <sub>v</sub>	1 + 0.19a <sub>x</sub>
"	" x 1 1/4	148 1/2	42.9	2.98	6.17	1.14	2.38	1 + 0.68a <sub>v</sub>	1 + 0.19a <sub>x</sub>
"	" x 1	128	36.9	2.90	6.00	1.14	2.36	1 + 0.72a <sub>v</sub>	1 + 0.20a <sub>x</sub>
"	" x 7/8	118	33.9	2.84	5.91	1.15	2.35	1 + 0.74a <sub>v</sub>	1 + 0.20a <sub>x</sub>
"	" x 3/4	107 1/2	30.9	2.77	5.82	1.16	2.35	1 + 0.78a <sub>v</sub>	1 + 0.20a <sub>x</sub>
"	" x 5/8	97 1/2	27.9	2.69	5.72	1.17	2.34	1 + 0.83a <sub>v</sub>	1 + 0.20a <sub>x</sub>
"	" x 1/2	87 1/2	24.9	2.58	5.61	1.18	2.35	1 + 0.90a <sub>v</sub>	1 + 0.21a <sub>x</sub>

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2 1/2 per cent. over this must be allowed. See page 7.

Each weight per foot is for the riveted shaft only. Weight of base, &c., to be added.

Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

W<sub>e</sub>=actual eccentric load; K=relative eccentricity coefficient; W<sub>c</sub>=equivalent concentric value; W<sub>c</sub>=W<sub>e</sub> x K.

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for a<sub>v</sub> and a<sub>x</sub> respectively.

For full explanations of tables, see notes commencing page 123 L.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND STANCHIONS.

Safe Concentric Loads, in Tons.  
Ends Fixed.

Reference Mark.	Size, D x B inches.	HEIGHTS IN FEET.												
		10	11	12	13	14	16	18	20	24	28	32	36	
102 K	13½ x 10	127	124	121	118	115	108	102	96	83	87	71	3	
101 K	13½ x "	113	110	107	104	102	96	290	584	773	261	6		
100 K	13 x "	99	596	994	291	688	983	678	372	962	351	7		
99 K	12½ x "	85	382	980	477	975	570	665	760	850	9			
92 K	14 x 14	433	427	421	414	408	395	382	369	344	318	293	267	
90 K	13½ x "	393	387	381	375	369	357	346	334	310	287	263	240	
88 K	13 x "	352	347	342	336	331	320	309	298	277	255	234	212	
86 K	12½ x 12	276	271	266	261	256	245	235	224	204	183	162	141	
84 K	12 x "	242	237	233	228	223	214	205	195	176	157	139	120	
83 K	11½ x "	225	221	216	212	207	198	189	180	162	145	127	109	
82 K	11½ x "	208	204	199	195	191	182	174	165	149	132	115	98	
81 K	11½ x "	191	187	183	179	175	167	159	151	134	118	102		
80 K	11 x "	173	170	166	162	158	151	143	135	120	105	90	2	
68 K	13 x 12	267	262	257	252	248	238	229	219	200	181	162	144	
66 K	12½ x "	233	228	224	220	216	207	199	190	173	157	140	123	
64 K	12 x "	199	195	191	187	184	176	169	161	146	131	116	102	
63 K	11½ x "	181	178	174	171	168	161	154	147	133	119	105	91	
62 K	11½ x "	164	161	158	155	151	145	138	132	119	106	93	80	
61 K	11½ x "	147	144	141	138	135	129	123	117	105	93	81	69	
60 K	11 x 10	114	111	108	105	102	96	089	983	771	459	1		
59 K	10½ x "	100	97	494	691	788	883	177	371	660	1			

Rivets ½-in. diam. at 6-in. pitch.

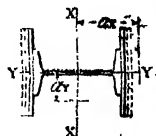
Rivets ½-in. diam. at 6-in. pitch.

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160.  
Safe loads are in accordance with the working stresses prescribed by the London County Council (General Powers) Act, 1900, for stanchions of mild steel having "both ends fixed."  
For other conditions and formulae, see notes commencing page 192 L.  
For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND STANCHIONS.

### Composition and Properties.



Composed of		Weight per foot in lbs.	Area in square inches.	Radii of Gyratlon.		Eccentricity Coefficients.			
One Steel Joist.	Plates, each flange to form.			Axis Y-Y	Axis X-X	Web.	Flange.	Axis Y-Y	Axis X-X
12 x 5	10 x 1/2	85 1/2	24.4	2.35	5.83	1.16	2.34	1 + 0.91av	1 + 0.20ax
"	" x 3/4	77	21.9	2.28	5.72	1.17	2.34	1 + 0.96av	1 + 0.20ax
"	" x 1	68 1/2	19.4	2.19	5.60	1.18	2.35	1 + 1.04av	1 + 0.21ax
"	" x 1 1/4	60	16.9	2.06	5.47	1.21	2.36	1 + 1.17av	1 + 0.21ax
10 x 8	14 x 2	264	76.6	3.59	5.57	1.17	2.58	1 + 0.55av	1 + 0.23ax
"	" x 1 1/2	240 1/2	69.6	3.54	5.42	1.17	2.55	1 + 0.56av	1 + 0.23ax
"	" x 1 1/4	216 1/2	62.6	3.48	5.27	1.18	2.52	1 + 0.58av	1 + 0.24ax
"	12 x 1 1/2	176	50.6	2.92	5.06	1.21	2.52	1 + 0.71av	1 + 0.25ax
"	" x 1	155 1/2	44.6	2.84	4.90	1.23	2.50	1 + 0.75av	1 + 0.25ax
"	" x 3/4	145	41.6	2.79	4.82	1.23	2.49	1 + 0.77av	1 + 0.25ax
"	" x 1/2	135	38.6	2.73	4.73	1.24	2.48	1 + 0.81av	1 + 0.26ax
"	" x 3/8	125	35.6	2.66	4.65	1.26	2.47	1 + 0.85av	1 + 0.26ax
"	" x 1/4	114 1/2	32.6	2.57	4.55	1.27	2.46	1 + 0.91av	1 + 0.27ax
10 x 6	12 x 1 1/2	167	48.3	3.06	5.39	1.13	2.45	1 + 0.64av	1 + 0.23ax
"	" x 1 1/4	146 1/2	42.3	3.00	5.24	1.13	2.42	1 + 0.67av	1 + 0.23ax
"	" x 1	126	36.3	2.92	5.08	1.14	2.40	1 + 0.70av	1 + 0.23ax
"	" x 3/4	116	33.3	2.87	5.00	1.15	2.38	1 + 0.73av	1 + 0.24ax
"	" x 1/2	105 1/2	30.3	2.80	4.91	1.15	2.37	1 + 0.76av	1 + 0.24ax
"	" x 3/8	95 1/2	27.3	2.72	4.82	1.16	2.36	1 + 0.81av	1 + 0.24ax
"	10 x 1 1/2	78 1/2	22.3	2.18	4.67	1.21	2.39	1 + 1.06av	1 + 0.25ax
"	" x 1	70	19.8	2.07	4.56	1.23	2.39	1 + 1.16av	1 + 0.26ax

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2 1/2 per cent. over this must be allowed. See page 7.

Each weight per foot is for the riveted shaft only. Weight of base, &c., to be added.

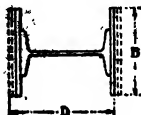
Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

We=actual eccentric load; K=relative eccentricity coefficient; Wc=equivalent concentric value; Wc=WxK.

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for a and ax respectively.

For full explanations of tables, see notes commencing page 192 I.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND STANCHIONS.

Safe Concentric Loads, in Tons.  
Ends Fixed.

Reference Mark.	Size, D x B inches.	HEIGHTS IN FEET.												
		10	11	12	13	14	16	18	20	24	28	32	36	
54 K	12 x 10	152	148	145	141	138	131	124	117	103	89	57	75	
53 K	11½ x "	138	135	132	128	125	119	112	106	93	28	26	7	
52 K	11½ x "	124	121	118	115	112	106	100	94	78	70	7		
51 K	11½ x 9	101	98	95	92	89	83	77	72	60	5			
50 K	11 x "	88	86	83	80	78	72	67	62	51	3			
49 K	10½ x "	76	73	71	68	66	60	55	50	41	7			
38 K	12 x 12	291	286	281	275	270	260	249	238	217	196	175	154	
36 K	11½ x "	257	253	248	243	238	229	219	209	190	171	152	132	
34 K	11 x "	223	219	215	210	206	197	189	180	163	146	128	111	
33 K	10½ x 10	180	176	171	167	163	154	145	136	119	101			
32 K	10½ x "	166	162	158	154	150	141	133	125	108	91	8		
31 K	10½ x "	152	148	144	141	137	129	121	113	97	82	1		
30 K	10 x "	138	135	131	127	123	116	109	101	86	72	0		
24 K	10 x 10	160	156	152	149	145	138	130	123	108	93	87	9	
23 K	9½ x "	146	142	139	135	132	125	118	111	98	84	47	0	
22 K	9½ x "	132	129	126	122	119	113	106	100	87	75	0		
21 K	9½ x 9	109	105	102	99	96	90	84	78	65	7			
20 K	9 x "	96	93	89	85	81	75	69	63	51	4			
19 K	8½ x "	84	81	78	75	72	66	60	54	42	3			
14 K	10 x 10	149	146	142	139	136	129	122	115	102	82	77	5	
13 K	9½ x "	135	132	129	126	123	117	110	104	92	79	56	9	
12 K	9½ x "	122	119	116	113	110	104	99	93	81	70	1	5	
11 K	9½ x 9	98	96	93	90	87	82	76	71	59	48	8		
10 K	9 x "	86	84	81	78	76	70	64	58	46	3			
9 K	8½ x "	74	71	69	67	64	60	55	50	41	5			

Rivets ½-in. diam. at 6-in. pitch.

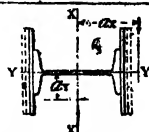
Rivets 3/4-in. diam. at 6-in. pitch.

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160.  
Safe loads are in accordance with the working stresses prescribed by the London County Council (General Powers) Act, 1900, for stanchions of mild steel having "both ends fixed."  
For other conditions and formulae, see notes commencing page 182 L.  
The safe load printed in italics is for a height greater than 90 ft.  
For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND STANCHIONS.

### Composition and Properties.



Composed of		Weight per foot in lbs.	Area in square inches.	Radii of Gyration.		Eccentricity Coefficients.			
One Steel Joist.	Plates, each flange to form.			Axis Y-Y	Axis X-X	Web.	Flange.	Axis Y-Y	Axis X-X
10 x 5	10 x 1	100½	28.8	2.47	5.11	1.15	2.38	1+0.82ax	1+0.23ax
"	" x 1½	92	26.3	2.43	5.02	1.15	2.37	1+0.85ax	1+0.23ax
"	" x 2	83½	23.8	2.38	4.93	1.16	2.36	1+0.89ax	1+0.24ax
"	9 x 2	70½	20.0	2.06	4.80	1.19	2.37	1+1.06ax	1+0.24ax
"	" x 2½	63	17.8	1.99	4.70	1.21	2.37	1+1.14ax	1+0.25ax
"	" x 3	55½	15.5	1.88	4.58	1.23	2.38	1+1.27ax	1+0.26ax
9 x 7	12 x 1½	184	53.0	3.00	4.81	1.19	2.56	1+0.67ax	1+0.26ax
"	" x 1½	164	47.0	2.94	4.66	1.19	2.53	1+0.70ax	1+0.27ax
"	" x 1	145½	41.0	2.85	4.50	1.20	2.50	1+0.74ax	1+0.27ax
"	10 x 1½	121½	34.5	2.35	4.36	1.25	2.52	1+0.90ax	1+0.28ax
"	" x 2	113	32.0	2.31	4.28	1.26	2.51	1+0.94ax	1+0.29ax
"	" x 2½	104½	29.5	2.25	4.19	1.27	2.50	1+0.98ax	1+0.29ax
"	" x 3	96	27.0	2.19	4.10	1.29	2.49	1+1.05ax	1+0.30ax
8 x 6	10 x 1	105½	30.3	2.47	4.13	1.18	2.47	1+0.82ax	1+0.29ax
"	" x 1½	97	27.8	2.42	4.05	1.19	2.45	1+0.85ax	1+0.30ax
"	" x 2	88½	25.3	2.37	3.97	1.20	2.43	1+0.89ax	1+0.30ax
"	9 x 2	75½	21.5	2.09	3.85	1.23	2.44	1+1.03ax	1+0.31ax
"	" x 2½	68	19.3	2.02	3.76	1.24	2.43	1+1.11ax	1+0.32ax
"	" x 3	60½	17.0	1.93	3.66	1.27	2.43	1+1.21ax	1+0.33ax
8 x 5	10 x 1	98½	28.2	2.50	4.19	1.14	2.43	1+0.80ax	1+0.29ax
"	" x 1½	90	25.7	2.46	4.11	1.15	2.41	1+0.83ax	1+0.29ax
"	" x 2	81½	23.2	2.41	4.03	1.15	2.39	1+0.86ax	1+0.29ax
"	9 x 2	69	19.5	2.10	3.91	1.18	2.40	1+1.02ax	1+0.30ax
"	" x 2½	61	17.2	2.03	3.82	1.19	2.39	1+1.10ax	1+0.31ax
"	" x 3	53½	15.0	1.93	3.72	1.21	2.38	1+1.21ax	1+0.32ax

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

Each weight per foot is for the riveted shaft only. Weights of base, &c., to be added.

Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

W = actual eccentric load; K = relative eccentricity coefficient; We = equivalent concentric value; We = W x K.

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for  $a_y$  and  $a_x$  respectively.

For full explanations of tables, see notes commencing page 182 L.

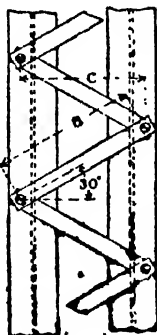
# REDPATH, BROWN & CO., LIMITED.



## COMPOUND STANCHIONS.

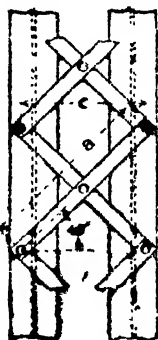
Safe Concentric Leads, in Tons.  
Ends Fixed.

Reference Mark.	Size, D x B inches.	HEIGHTS IN FEET.											
		10	12	14	16	18	20	22	24	28	32	36	40
30 L	24 x 26	363	359	355	352	348	344	340	336	329	321	314	306
29 L	20 x 23	320	316	312	308	304	300	296	292	284	276	268	260
28 L	18 x 21	268	264	260	257	253	249	246	242	235	227	220	212
27 L	16 x 18	219	215	211	208	204	201	197	194	186	179	172	165
26 L	15 x 17	207	203	199	196	192	188	184	181	173	166	159	151
25 L	15 x 16	147	144	142	139	136	134	131	128	123	118	112	107
24 L	14 x 17	200	196	193	189	185	182	178	175	168	160	153	146
23 L	14 x 17	161	158	155	152	150	147	144	141	135	129	124	118

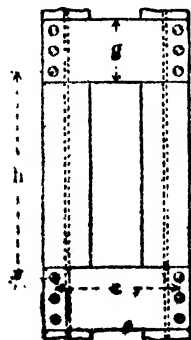


SINGLE LATTICING.

Suitable for values of  $c$ , not exceeding 15 inches.



DOUBLE LATTICING.



BATTEN PLATE.

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160.

Safe loads are in accordance with the working stresses prescribed by the London County Council (General Powers) Act, 1909, for stanchions of mild steel having "both ends fixed"

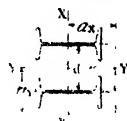
For other conditions and formulae, see notes commencing page 198 L.

For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND STANCHIONS.

Composition and Properties.



Composed of two Steel Joists Latticed.	Weight per foot in lbs.	Area in square inches.	d. Centres of Webs. Inches.	Radii of Gyration.		Eccentricity Coefficients.			
				Axis Y-Y	Axis X-X	Web.	Flange.	Axis Y-Y	Axis X-X
24 x 7½	200	58.8	15.5	9.37	9.50	2.42	2.60	1 + 0.15av	1 + 0.13ax
20 x 7½	178	52.3	15.5	7.90	7.99	2.48	2.57	1 + 0.19av	1 + 0.18ax
18 x 7	150	44.1	14.0	7.15	7.22	2.50	2.56	1 + 0.21av	1 + 0.17ax
16 x 6	124	36.1	12.0	6.12	6.31	2.51	2.61	1 + 0.24av	1 + 0.20ax
15 x 6	118	34.7	11.0	5.64	6.02	2.53	2.55	1 + 0.27av	1 + 0.21ax
15 x 5	84	24.7	11.0	5.59	5.89	2.47	2.62	1 + 0.26av	1 + 0.22ax
14 x 6a	114	33.5	11.0	5.65	5.61	2.53	2.54	1 + 0.27av	1 + 0.22ax
14 x 6b	92	27.0	11.0	5.64	5.70	2.52	2.51	1 + 0.27av	1 + 0.22ax

CONVENTIONAL MAXIMUM SPACING AND MINIMUM PROPORTIONS OF LATTICE BARS AND BATTEN PLATES FOR CONCENTRIC LOADING (*Am. Ry. Engineering and Maintenance of Way Assoc.*).

Width of Joist Flange. Inches.	7½	7	6	5
Width of Lattice Bar. Inches.	2½	2½	2½	2½
Diameter of Rivet.	¾	¾	¾	¾

### SINGLE LATTICING -

Maximum angle of inclination with horizontal = 33 degrees.  
Minimum thickness = 1/40th of  $a$ , the diagonal centres of rivets.  
Maximum horizontal centres of rivets,  $c$  = 10 inches.

### DOUBLE LATTICING -

Maximum angle of inclination with horizontal = 45 degrees.  
Minimum thickness = 1/60th of  $a$ , the diagonal centres of rivets.

### BATTEN PLATES -

Maximum centres of end rivets of batten plates =  $h$  inches.  
Let  $l$  = height of stanchion in inches, and  $h$  = radius of gyration of one joist.  
Then  $h$  =  $\frac{l}{10}$  least.

Minimum thickness = 1/80th of  $c$ , the horizontal centres of rivets.  
Minimum width  $g$  =  $c$ , the horizontal centres of rivets for end plates.  
" "  $g$  =  $\frac{1}{2}c$ , " " intermediate plates.

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 1.

Each weight per foot is for the shaft only. Weights of lattices, batten, &c., to be added.

Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

We = actual eccentric load; K = relative eccentricity coefficient; We = equivalent concentric value; We = We x K.

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for  $av$  and  $ax$  respectively.

For full explanations of tables, see notes commencing page 122 L.

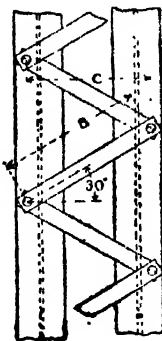
# REDPATH, BROWN & CO., LIMITED.



## COMPOUND STANCHIONS.

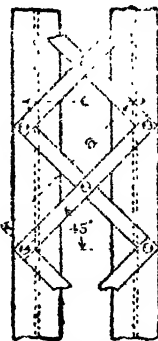
Safe Concentric Loads, in Tons.  
Ends Fixed.

Reference Mark.	Size, D x B inches.	HEIGHTS IN FEET.											
		10	12	14	16	18	20	22	24	28	32	36	40
22 L	12 x 15	186	182	178	174	170	165	161	157	149	141	133	125
21 L	12 x 15	151	148	145	141	138	135	131	128	121	115	108	102
20 L	12 x 14	110	107	105	102	100	97	95	92	88	83	78	73
18 L	10 x 13	140	137	133	129	125	121	117	113	105	97	89	81
17 L	10 x 12	100	97	94	91	88	85	82	79	74	68	62	56
15 L	9 x 11	70	67	65	63	61	59	57	55	51	47	43	39
14 L	8 x 12	114	111	107	103	99	96	92	88	81	73	65	58
13 L	8 x 11	91	88	85	82	79	76	73	70	66	61	55	49

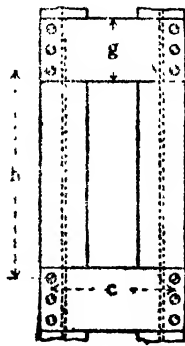


SINGLE LATTICING.

Suitable for values of  $c$ , not exceeding 15 inches.



DOUBLE LATTICING



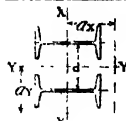
BATTEN PLATES.

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160.  
Safe loads are in accordance with the working stresses prescribed by the London County Council (General Powers) Act, 1900, for stanchions of mild steel having "both ends fixed."  
For other conditions and formulae, see notes commencing page 192 L.  
Safe loads printed in italics are for heights greater than 400.  
For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND STANCHIONS.

Composition and Properties.



Composed of two Steel Joists Latticed.	Weight per foot in lbs.	Area in square inches.	d, Centre of Web, inches.	Radii of Gyration.		Eccentricity Coefficients.			
				Axis Y-Y	Axis X-X	Web.	Flange	Axis Y-Y	Axis X-X
12 x 6a	108	31.7	9.6	4.69	4.86	2.62	3.52	1 + 0.34a	1 + 0.26ax
12 x 6b	88	25.9	9.9	4.68	4.03	2.61	2.48	1 + 0.34a	1 + 0.25ax
12 x 5	61	18.8	9.4	4.61	4.53	2.54	2.54	1 + 0.33a	1 + 0.26ax
10 x 6	84	24.7	7.0	3.75	4.14	2.71	2.46	1 + 0.46a	1 + 0.29ax
10 x 5	60	17.6	7.3	3.55	4.06	2.65	2.52	1 + 0.45a	1 + 0.30ax
9 x 4	42	12.3	6.0	3.59	3.62	2.55	2.51	1 + 0.43a	1 + 0.34ax
8 x 6	70	20.6	6.0	3.27	3.28	2.80	2.49	1 + 0.56a	1 + 0.37ax
8 x 5	56	16.5	6.0	3.20	3.29	2.71	2.48	1 + 0.54a	1 + 0.37ax

CONVENTIONAL MAXIMUM SPACING AND MINIMUM PROPORTIONS OF LATTICE BARS AND BATTEN PLATES FOR CONCENTRIC LOADING (Am. Ry. Engineering and Maintenance of Way Assoc.).

Width of Joist Flange Inches	6	4
Width of Lattice Bar Inches	2	2
Diameter of Rivets	1	5/8

### SINGLE LATTICING

Maximum angle of inclination with horizontal - 45 degrees  
Minimum thickness = 1/40th of  $\alpha$ , the horizontal centre of rivets.  
Maximum horizontal centres of rivets,  $c = 12$  inches.

### DOUBLE LATTICING

Maximum angle of inclination with horizontal - 45 degrees  
Maximum thickness = 1/80th of  $\alpha$ , the diagonal centres of rivets.

### BATTEN PLATES

Maximum centres of end rivets of batten plates = 8 inches  
Let  $l$  = height of stanchion in inches, and  $k$  = radius of gyration of one joist.  
Then  $h = k \times k$  least.

Then  $h = k \times k$  least.

Minimum thickness = 1/50th of  $c$ , the horizontal centres of rivets.

Minimum width  $g = c$ , the horizontal centres of rivets for end plates.

" "  $g = 4c$ , " " " intermediate plates.

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 5 per cent. over this must be allowed. See page 7.

Each weight per foot is for the shaft only. Weights of lattices, batten, &c., to be added.

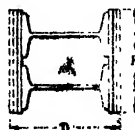
Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

We = actual eccentric load; R = relative eccentricity coefficient; We = equivalent concentric value; We = We x R.

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for  $d_y$  and  $d_x$  respectively.

For full explanations of tables, see notes commencing page 122 L.

## REDPATH, BROWN &amp; CO., LIMITED.



## COMPOUND STANCHIONS.

Safe Concentric Loads, in Tons.  
Ends Fixed.

Reference Mark.	Span in Feet	HEIGHTS IN FEET											
		10	12	14	16	18	20	22	24	28	32	36	40
282 M	18 x 24	960	741	608	511	489	466	442	438	410	382	354	327
280 M	27 x 24	867	680	537	454	411	398	386	373	347	321	295	269
278 M	27 x 24	790	618	496	415	373	351	339	327	303	276	250	224
276 M	26 x 24	717	561	456	385	343	321	309	297	273	246	220	194
274 M	26 x 24	645	505	415	345	303	281	269	257	233	206	180	154
273 M	25 x 20	557	447	367	307	265	243	231	219	195	168	142	116
272 M	25 x 20	527	418	338	278	236	214	202	190	166	139	113	87
271 M	25 x 18	478	368	288	228	186	164	152	140	116	89	63	37
262 M	22 x 24	867	680	537	454	411	398	386	373	347	321	295	269
260 M	22 x 24	824	641	508	425	382	369	357	345	319	293	267	241
258 M	22 x 24	751	588	465	382	339	326	314	302	276	250	224	198
256 M	22 x 20	698	545	432	349	306	284	272	260	234	208	182	156
254 M	22 x 20	649	506	403	320	277	255	243	231	205	179	153	127
253 M	21 x 20	519	399	306	243	200	178	166	154	128	102	76	50
252 M	21 x 20	489	369	276	213	170	148	136	124	100	74	48	22
251 M	21 x 18	440	331	248	185	142	120	108	96	72	46	20	14
242 M	22 x 18	685	531	418	335	292	270	258	246	220	194	168	142
240 M	21 x 18	632	489	376	293	250	228	216	204	178	152	126	100
238 M	21 x 18	578	445	332	249	206	184	172	160	134	108	82	56
236 M	20 x 18	525	404	291	208	165	143	131	119	93	67	41	15
234 M	20 x 18	472	362	249	166	123	101	89	77	51	25	19	13
233 M	19 x 16	419	309	196	113	70	48	36	24	12	1	1	1
232 M	19 x 16	396	286	173	90	47	25	13	1	1	1	1	1
231 M	19 x 16	372	262	149	76	33	11	1	1	1	1	1	1

Rivets 2-in. diam. at 6-in. pitch.

The above safe loads are for endways of stanchions up to, but not exceeding 160 feet. Safe loads are in accordance with the working stresses prescribed by the London County Council (General Powers) Act, 1906, for stanchions of mild steel having both ends fixed.

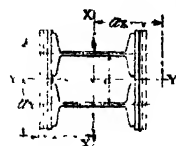
For other conditions and formulae, see notes commencing page 1921.

For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND STANCHIONS.

### Composition and Properties.



Composed of		Weight per foot in lbs.	Area of web in sq. in.	Centre of web in inches.	Radii of gyration		Eccentricity Coefficients.			
Two Steel Joists.	Plates each flange to form				X <sub>1</sub> Y	X <sub>2</sub> X	Web	Flange	Axis Y-Y	Axis X-X
24 x 7 1/2	21 x 2	530	151.8	12	6.65	11.50	2.71	2.41	1 + 0.27a <sub>v</sub>	1 + 0.10a <sub>x</sub>
"	" x 1 1/2	489	112.8	"	6.63	11.61	2.72	2.40	1 + 0.27a <sub>v</sub>	1 + 0.10a <sub>x</sub>
"	" x 1 1/4	448 1/2	130.8	"	6.60	11.41	2.74	2.40	1 + 0.28a <sub>v</sub>	1 + 0.11a <sub>x</sub>
"	" x 1 1/4	407 1/2	118.8	"	6.57	11.12	2.75	2.40	1 + 0.28a <sub>v</sub>	1 + 0.11a <sub>x</sub>
"	" x 1	364	106.8	"	6.53	10.95	2.78	2.41	1 + 0.28a <sub>v</sub>	1 + 0.11a <sub>x</sub>
"	20 x 2	322 1/2	93.8	10	5.43	10.69	2.80	2.45	1 + 0.34a <sub>v</sub>	1 + 0.11a <sub>x</sub>
"	" x 2	305 1/2	88.8	"	5.41	10.56	2.81	2.46	1 + 0.34a <sub>v</sub>	1 + 0.12a <sub>x</sub>
"	18 x 2	280	81.3	9	4.87	10.36	2.82	2.48	1 + 0.38a <sub>v</sub>	1 + 0.12a <sub>x</sub>
20 x 7 1/2	21 x 2	508	118.3	12	6.68	10.95	2.70	2.43	1 + 0.27a <sub>v</sub>	1 + 0.12a <sub>x</sub>
"	" x 1 1/2	467 1/2	136.3	"	6.65	9.87	2.71	2.42	1 + 0.27a <sub>v</sub>	1 + 0.12a <sub>x</sub>
"	" x 1 1/4	426 1/2	124.3	"	6.63	9.69	2.72	2.41	1 + 0.27a <sub>v</sub>	1 + 0.12a <sub>x</sub>
"	20 x 1 1/2	351 1/2	102.3	10	5.50	9.35	2.75	2.44	1 + 0.33a <sub>v</sub>	1 + 0.13a <sub>x</sub>
"	" x 1 1/4	318	92.3	"	5.17	9.69	2.77	2.44	1 + 0.34a <sub>v</sub>	1 + 0.13a <sub>x</sub>
"	" x 1	300 1/2	87.3	"	5.15	9.69	2.78	2.45	1 + 0.34a <sub>v</sub>	1 + 0.13a <sub>x</sub>
"	" x 1	283 1/2	82.3	"	5.46	8.93	2.80	2.45	1 + 0.34a <sub>v</sub>	1 + 0.14a <sub>x</sub>
"	18 x 2	258	74.3	9	4.89	8.75	2.81	2.47	1 + 0.36a <sub>v</sub>	1 + 0.14a <sub>x</sub>
18 x 7	18 x 2	398 1/2	116.1	9	5.02	8.07	2.71	2.48	1 + 0.36a <sub>v</sub>	1 + 0.14a <sub>x</sub>
"	" x 1 1/2	368	107.1	"	5.01	8.88	2.72	2.47	1 + 0.36a <sub>v</sub>	1 + 0.14a <sub>x</sub>
"	" x 1 1/4	337 1/2	98.1	"	4.99	8.71	2.73	2.46	1 + 0.36a <sub>v</sub>	1 + 0.14a <sub>x</sub>
"	" x 1 1/4	307	89.1	"	4.97	8.52	2.74	2.45	1 + 0.37a <sub>v</sub>	1 + 0.14a <sub>x</sub>
"	" x 1	276 1/2	80.1	"	4.94	8.32	2.76	2.45	1 + 0.37a <sub>v</sub>	1 + 0.15a <sub>x</sub>
"	16 x 2	249	72.1	8	4.40	8.15	2.77	2.47	1 + 0.42a <sub>v</sub>	1 + 0.15a <sub>x</sub>
"	" x 2	235 1/2	68.1	"	4.38	8.01	2.78	2.47	1 + 0.42a <sub>v</sub>	1 + 0.15a <sub>x</sub>
"	" x 2	222	64.1	"	4.37	7.93	2.79	2.47	1 + 0.42a <sub>v</sub>	1 + 0.16a <sub>x</sub>

In each case the weight per foot given is the minimum that can be relied on, and a rolling margin of 21 per cent. over this must be allowed. See page 7.

Each weight per foot is for the riveted shaft only. Weight of base A<sub>v</sub> & B<sub>v</sub> to be added.

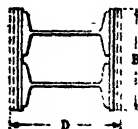
Least radii of gyration, and relative eccentricity coefficients are printed in parenthesis (p. 3).

W<sub>a</sub> = actual eccentricity load; K = relative eccentricity coefficient; W<sub>e</sub> = equivalent eccentricity value; W<sub>ec</sub> = W<sub>a</sub> x K.

In axial eccentricity coefficient substitute actual value of "arm of eccentricity" for A<sub>v</sub> and A<sub>x</sub> respectively.

For full explanations of tables, see notes commencing page 192 L.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND STANCHIONS.

Safe Concentric Loads, in Tons.  
Ends Fixed.

Reference Mark.	Size, D x B inches.	HEIGHTS IN FEET.											
		10	12	14	16	18	20	22	24	28	32	36	40
218 M	16 x 16	491	480	469	457	446	434	423	411	389	366	343	320
216 M	18½ x "	447	441	434	425	413	398	382	372	351	330	309	289
214 M	18 x "	398	388	379	370	360	351	342	332	314	295	276	257
213 M	17½ x 14	518	509	500	490	480	469	459	448	426	403	380	357
212 M	17½ x "	328	319	310	301	292	283	274	265	247	229	211	193
211 M	17½ x "	308	299	291	282	274	265	257	249	232	215	198	181
210 M	17 x "	288	280	272	264	256	248	240	232	216	200	194	168
198 M	18 x 16	481	470	459	448	437	425	414	403	381	358	336	314
196 M	17½ x "	434	424	414	404	394	384	374	364	343	323	303	283
194 M	17 x "	388	378	369	360	351	342	333	324	306	288	269	251
193 M	16½ x 14	508	500	490	481	471	462	453	444	426	408	390	372
192 M	16½ x "	318	309	301	292	283	275	266	257	240	222	205	188
191 M	16½ x "	298	290	282	273	265	257	249	241	224	208	192	175
190 M	16 x "	278	270	262	255	247	240	232	224	209	194	178	163
184 M	17 x 14	501	493	485	476	468	459	451	444	427	411	394	378
183 M	16½ x "	281	273	265	258	250	242	235	227	212	196	181	166
182 M	16½ x "	261	254	246	239	232	225	218	211	196	182	168	153
181 M	16½ x "	241	234	227	221	214	207	201	194	181	167	154	141
180 M	16 x "	220	214	208	202	196	190	184	178	165	153	141	128

Rivets ½-in. diam. at 6-in. pitch.

Rivets ¾-in. diam. at 6-in. pitch.

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160.

Safe loads are in accordance with the working stresses prescribed by the London County Council (General Powers) Act, 1909, for stanchions of mild steel having "both ends fixed."

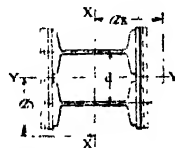
For other conditions and formulae, see notes commencing page 192 L.

For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND STANCHIONS.

### Composition and Properties.



Composed of		Weight per foot in lbs.	Area in square inches.	d. Centres of Webs in inches.	Radii of Gyration.		Eccentricity Coefficients.			
Two Steel Joists.	Plates, each flange to form.				Axis Y-Y	Axis X-X	Web.	Flange.	Axis Y-Y	Axis X-X
16 x 6	16 x 1 1/2	285 1/2	84.4	8	4.42	7.80	2.74	2.49	1+0.41ax	1+0.16ax
"	" x 1 1/2	262 1/2	76.4	"	4.41	7.61	2.76	2.48	1+0.41ax	1+0.16ax
"	" x 1 1/2	235	68.4	"	4.39	7.42	2.78	2.47	1+0.42ax	1+0.17ax
"	11 x 3/4	210	60.0	7	3.81	7.21	2.79	2.50	1+0.48ax	1+0.17ax
"	" x 3/4	198	57.4	"	3.83	7.13	2.80	2.51	1+0.48ax	1+0.17ax
"	" x 3/4	186	53.9	"	3.82	7.02	2.81	2.51	1+0.48ax	1+0.18ax
"	" x 3/4	174	50.3	"	3.80	6.90	2.83	2.52	1+0.49ax	1+0.18ax
15 x 6	16 x 1 1/2	283 1/2	82.7	8	4.41	7.40	2.72	2.48	1+0.41ax	1+0.17ax
"	" x 1 1/2	256 1/2	74.7	"	4.43	7.23	2.74	2.47	1+0.41ax	1+0.17ax
"	" x 1 1/2	229 1/2	66.7	"	4.40	7.04	2.75	2.46	1+0.41ax	1+0.17ax
"	14 x 3/4	204	59.2	7	3.86	6.88	2.76	2.48	1+0.47ax	1+0.19ax
"	" x 3/4	192	55.7	"	3.84	6.78	2.78	2.48	1+0.48ax	1+0.18ax
"	" x 3/4	180	52.2	"	3.83	6.67	2.79	2.48	1+0.48ax	1+0.18ax
"	" x 3/4	168	48.7	"	3.82	6.56	2.80	2.49	1+0.48ax	1+0.19ax
15 x 5	14 x 1 1/2	181 1/2	52.7	7	3.85	7.09	2.75	2.44	1+0.47ax	1+0.17ax
"	" x 3/4	170	49.2	"	3.84	6.98	2.76	2.44	1+0.48ax	1+0.17ax
"	" x 3/4	158	45.7	"	3.82	6.87	2.78	2.44	1+0.48ax	1+0.18ax
"	" x 3/4	146	42.2	"	3.81	6.75	2.79	2.45	1+0.48ax	1+0.18ax
"	" x 3/4	134	38.7	"	3.78	6.62	2.81	2.46	1+0.49ax	1+0.18ax

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2 1/2 per cent. over this must be allowed. See page 7.

Each weight per foot is for the riveted shaft only. Weight of base, &c., to be added.

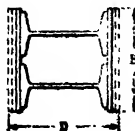
Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

We=actual eccentric load; K=relative eccentricity coefficient; We=equivalent concentric value; Wc=WxK.

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for ax and ax' respectively.

For full explanations of tables, see notes commencing page 192 L.

## REDPATH, BROWN &amp; CO., LIMITED.



## COMPOUND STANCHIONS.

Safe Concentric Loads, in Tons.  
Ends Fixed.

Reference Mark.	Size, D x B inches	HEIGHTS IN FEET.											
		10	12	14	16	18	20	22	24	28	32	36	40
168 M	17 x 16	175	461	173	442	131	129	369	398	376	354	332	310
166 M	16 1/2 x "	174	418	168	348	388	378	368	358	338	318	298	279
164 M	16 x "	381	372	363	354	345	336	327	319	301	283	265	247
163 M	15 1/2 x 11	322	328	314	305	296	287	278	269	251	233	215	197
162 M	15 1/4 x "	312	308	295	286	278	269	261	252	235	218	201	184
161 M	15 1/2 x "	291	283	275	267	259	251	243	235	220	204	188	172
160 M	15 x "	271	263	256	248	241	233	226	219	204	189	174	159
148 M	17 x 16	137	127	117	107	97	87	77	66	54	42	30	28
146 M	16 1/2 x "	350	381	372	363	354	345	336	327	309	291	273	255
144 M	16 x "	343	351	347	345	341	336	330	327	311	295	279	263
143 M	15 1/2 x 11	295	287	279	271	263	255	247	239	223	207	191	175
142 M	15 1/4 x "	275	267	260	252	245	237	230	222	207	193	178	163
141 M	15 1/2 x "	254	248	241	234	227	220	213	206	192	178	164	150
140 M	15 x "	234	228	221	215	208	202	196	189	176	164	151	138
128 M	15 x 14	122	111	100	89	77	66	55	43	32	20	27	25
126 M	14 1/2 x "	382	372	362	351	341	331	321	310	290	269	249	228
124 M	14 x "	342	333	323	314	305	296	286	277	259	240	222	203
123 M	13 1/2 x "	322	313	304	295	287	278	269	261	243	226	208	191
122 M	13 x "	302	293	285	277	269	261	252	244	228	211	195	179
121 M	13 1/4 x "	281	274	266	258	251	243	235	228	212	197	182	166
120 M	13 x "	261	251	247	240	233	225	218	211	197	182	168	154

Rivets 1/2 in. diam. at 6-in. pitch.

The above safe loads are calculated for ratios of slenderness up to, but not exceeding 160.

Safe loads are in accordance with the working stresses prescribed by the London County Council (General Powers) Act, 1900, for stanchions of mild steel having "both ends fixed."

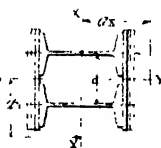
For other conditions and formulae, see notes commencing page 192 L.

For explanations of properties, &amp;c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND STANCHIONS.

### Composition and Properties.



Composed of		Weight per foot in lbs	Area in square inches	d Centres of Webs in inches	Radii of Gyration.		Eccentricity Coefficients.			
Two Steel Joists.	Plates each flange to form				Axis Y-Y	Axis X-X	Web.	Flange.	Axis Y-Y	Axis X-X
14 x 6a	16 x 1 1/2	279 1/2	81.5	8	4.45	6.97	2.72	2.49	1+0.41a <sub>v</sub>	1+0.18a <sub>x</sub>
"	" x 1 1/4	252 1/2	73.6	"	4.43	6.79	2.73	2.48	1+0.41a <sub>v</sub>	1+0.18a <sub>x</sub>
"	" x 1	225 1/2	65.5	"	4.41	6.61	2.75	2.46	1+0.41a <sub>v</sub>	1+0.18a <sub>x</sub>
"	11 x 3/4	200	58.0	7	3.86	6.43	2.76	2.49	1+0.47a <sub>v</sub>	1+0.19a <sub>x</sub>
"	" x 3/8	188	54.5	"	3.85	6.36	2.77	2.48	1+0.47a <sub>v</sub>	1+0.19a <sub>x</sub>
"	" x 3/8	176	51.0	"	3.84	6.26	2.78	2.48	1+0.48a <sub>v</sub>	1+0.20a <sub>x</sub>
"	" x 3/8	164	47.5	"	3.82	6.15	2.80	2.49	1+0.48a <sub>v</sub>	1+0.20a <sub>x</sub>
14 x 6b	16 x 1 1/4	257 1/2	75.0	8	4.47	7.09	2.68	2.44	1+0.40a <sub>v</sub>	1+0.17a <sub>x</sub>
"	" x 1 1/4	230 1/2	67.0	"	4.45	6.92	2.70	2.42	1+0.41a <sub>v</sub>	1+0.17a <sub>x</sub>
"	" x 1	203 1/2	59.0	"	4.43	6.74	2.72	2.41	1+0.41a <sub>v</sub>	1+0.18a <sub>x</sub>
"	14 x 3/4	178	51.5	7	3.87	6.49	2.73	2.43	1+0.47a <sub>v</sub>	1+0.18a <sub>x</sub>
"	" x 3/8	166	48.0	"	3.86	6.40	2.74	2.43	1+0.47a <sub>v</sub>	1+0.19a <sub>x</sub>
"	" x 3/8	154	44.5	"	3.85	6.38	2.75	2.43	1+0.47a <sub>v</sub>	1+0.19a <sub>x</sub>
"	" x 3/8	142	41.0	"	3.83	6.27	2.76	2.43	1+0.48a <sub>v</sub>	1+0.19a <sub>x</sub>
12 x 6a	14 x 1 1/4	233 1/2	73.7	7	3.91	5.92	2.71	2.35	1+0.46a <sub>v</sub>	1+0.21a <sub>x</sub>
"	" x 1 1/4	229 1/2	66.7	"	3.90	5.86	2.72	2.33	1+0.46a <sub>v</sub>	1+0.21a <sub>x</sub>
"	" x 1	207 1/2	59.7	"	3.88	5.69	2.74	2.51	1+0.47a <sub>v</sub>	1+0.22a <sub>x</sub>
"	" x 3/4	191	56.2	"	3.87	5.60	2.75	2.51	1+0.47a <sub>v</sub>	1+0.22a <sub>x</sub>
"	" x 3/8	182	52.7	"	3.86	5.51	2.76	2.50	1+0.47a <sub>v</sub>	1+0.22a <sub>x</sub>
"	" x 3/8	170	49.2	"	3.85	5.42	2.77	2.49	1+0.47a <sub>v</sub>	1+0.23a <sub>x</sub>
"	" x 3/8	158	45.7	"	3.84	5.32	2.78	2.49	1+0.48a <sub>v</sub>	1+0.23a <sub>x</sub>

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2 1/2 per cent. over this must be allowed. See page 7.

Each weight per foot is for the riveted shaft only. Weight of base, &c., to be added.

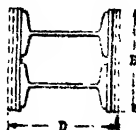
Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

We = actual eccentric load, K = relative eccentricity coefficient; We = equivalent concentric value; Wc = We x K.

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for a<sub>v</sub> and a<sub>x</sub> respectively.

For full explanations of tables, see notes commencing page 192 L.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND STANCHIONS.

Safe Concentric Loads, in Tons.  
Ends Fixed.

Reference Mark.	Size D x B inches.	HEIGHTS IN FEET.												
		10	12	14	16	18	20	22	24	28	32	36	40	
108 M	15 x 14	580	370	368	358	317	337	327	316	296	275	254	233	
106 M	14 x " "	410	335	330	321	311	302	293	283	265	246	227	209	
104 M	14 x " "	308	300	292	284	275	267	259	250	231	217	201	184	
103 M	13 1/2 x " "	288	280	273	265	257	249	241	234	218	203	187	172	
102 M	13 x " "	208	201	193	186	178	172	164	157	169	158	144	129	
101 M	13 x " "	148	141	134	128	121	114	107	101	117	104	90	77	
100 M	13 x " "	228	221	215	209	203	197	190	184	172	159	147	134	
99 M	12 1/2 x " "	208	202	195	190	185	179	173	168	156	145	133	122	
94 M	14 x 12	230	232	224	216	209	201	193	185	170	155	139	124	
93 M	13 1/2 x " "	222	215	208	201	194	187	179	172	158	143	129	115	
92 M	13 1/2 x " "	205	199	192	185	179	172	166	159	146	132	119	106	
91 M	13 x " "	189	182	176	170	164	158	152	146	133	121	109	96	
90 M	13 x " "	172	166	160	155	149	144	138	132	121	110	99	87	
89 M	12 1/2 x " "	155	150	145	139	134	129	124	119	109	99	88	78	
78 M	13 x 14	382	372	362	352	342	331	321	311	291	270	250	230	
76 M	12 1/2 x " "	342	335	324	315	305	296	287	278	260	242	223	205	
74 M	12 x " "	302	294	286	277	269	261	253	245	229	213	196	180	
73 M	11 1/2 x " "	282	274	266	259	251	244	236	229	213	198	183	168	
72 M	11 1/2 x " "	261	254	247	240	233	226	219	212	198	184	170	156	
71 M	11 1/2 x " "	241	235	228	222	215	209	202	195	182	169	156	143	
70 M	11 x " "	221	215	209	203	197	191	185	179	167	155	143	131	
69 M	10 1/2 x " "	201	195	190	184	179	173	168	162	151	140	129	118	

Rivets 3/4 in diam at 6-in. pitch.

Rivets 1/2 in diam at 6-in. pitch.

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160.

Safe loads are in accordance with the working stresses prescribed by the London County Council (General Powers) Act, 1900, for stanchions of mild steel having "both ends fixed."

For other conditions and formulae, see notes commencing page 192 L.

Safe loads printed in italics are for heights greater than 40 ft.

For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND STANCHIONS.

### Composition and Properties.



Composed of		Weight per foot in lbs.	Area in square inches.	d. Centres of Webs in inches.	Radii of Gyration		Eccentricity Coefficients.			
Two Steel Joists.	Plates, each flange to form.				Axis Y-Y	Axis X-X	Web.	Flange.	Axis Y-Y	Axis X-X
12 x 6	14 x 1	233 1/2	67.9	7	3.93	5.15	2.68	2.36	1	0.46ax, 1 + 0.20ax
"	" x 1	209 1/2	60.9	"	3.91	5.97	2.69	2.48	1	0.46ax, 1 + 0.21ax
"	" x 1	185 1/2	53.9	"	3.90	5.80	2.71	2.40	1	0.46ax, 1 + 0.21ax
"	" x 2	174	50.4	"	3.89	5.72	2.72	2.35	1	0.47ax, 1 + 0.21ax
"	" x 2	162	46.9	"	3.87	5.6	2.73	2.31	1	0.47ax, 1 + 0.22ax
"	" x 2	150	43.4	"	3.86	5.53	2.74	2.28	1	0.47ax, 1 + 0.22ax
"	" x 2	140	39.9	"	3.84	5.45	2.75	2.25	1	0.48ax, 1 + 0.22ax
"	" x 2	126	36.4	"	3.83	5.32	2.77	2.20	1	0.48ax, 1 + 0.23ax
12 x 5	12 x 1	148	42.8	"	3.83	5.80	2.71	2.41	1	0.54ax, 1 + 0.21ax
"	" x 1	135	39.8	"	3.82	5.71	2.72	2.34	1	0.54ax, 1 + 0.21ax
"	" x 1	127 1/2	36.8	"	3.81	5.64	2.73	2.31	1	0.55ax, 1 + 0.21ax
"	" x 1	117 1/2	33.8	"	3.80	5.57	2.75	2.28	1	0.55ax, 1 + 0.22ax
"	" x 1	107 1/2	30.8	"	3.78	5.43	2.76	2.25	1	0.56ax, 1 + 0.22ax
"	" x 1	97	27.8	"	3.76	5.31	2.79	2.19	1	0.56ax, 1 + 0.23ax
10 x 6	14 x 1 1/2	229 1/2	66.7	7	3.94	5.22	2.67	2.35	1	0.45ax, 1 + 0.24ax
"	" x 1 1/2	205 1/2	59.7	"	3.92	5.07	2.68	2.32	1	0.46ax, 1 + 0.25ax
"	" x 1 1/2	181 1/2	52.7	"	3.91	4.91	2.70	2.29	1	0.46ax, 1 + 0.25ax
"	" x 1 1/2	170	49.2	"	3.90	4.83	2.71	2.18	1	0.46ax, 1 + 0.25ax
"	" x 1 1/2	158	45.7	"	3.89	4.75	2.71	2.17	1	0.46ax, 1 + 0.26ax
"	" x 1 1/2	146	42.2	"	3.87	4.69	2.73	2.11	1	0.47ax, 1 + 0.26ax
"	" x 1 1/2	134	38.7	"	3.86	4.57	2.74	2.10	1	0.47ax, 1 + 0.27ax
"	" x 1 1/2	122	35.2	"	3.84	4.47	2.76	2.14	1	0.48ax, 1 + 0.27ax

In each case the weight per foot given is the minimum that can be relied, and a rolling margin of 24 per cent. over this must be allowed. See p. 87.

Each weight per foot is for the riveted shaft only. Weight of rivets to be added.

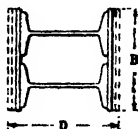
Least radii of gyration and relative eccentricity coefficients are indicated in prominent type.

We = actual eccentric load; K = relative eccentricity coefficient; We = equivalent concentric value;  $We = We \times K$ .

In axial eccentricity coefficients substitute actual value of "axis of eccentricity," for  $ax$  and  $ax$  respectively.

For full explanations of tables, see notes commencing page 192 L.

## REDPATH, BROWN &amp; CO., LIMITED.



## COMPOUND STANCHIONS.

Safe Concentric Loads, in Tons.  
Ends Fixed.

Reference Mark.	Size, D x B inches.	HEIGHTS IN FEET.											
		10	12	14	16	18	20	22	24	28	32	36	40
64 M	12 x 12	233	225	218	210	203	195	188	181	166	151	136	121
63 M	11½ x "	216	209	202	195	188	181	174	167	153	139	126	112
62 M	11¼ x "	199	193	186	180	173	167	160	154	141	128	115	103
61 M	11½ x "	182	176	170	164	158	153	147	141	129	117	105	94.0
60 M	11 x "	165	160	154	149	144	138	133	127	117	106	95.6	84.8
59 M	10½ x "	148	143	138	134	129	124	119	114	104	95	85.5	75.6
50 M	10 x 10	120	116	111	106	101	96.5	91.6	86.7	78.9	71.1	64.2	57.4
49 M	9½ x "	107	102	98.5	94.9	91.3	87.5	83.7	79.7	71.6	63.9	57.2	50.5
38 M	11 x 11	359	349	340	330	321	311	302	292	273	254	235	216
36 M	10½ x "	318	310	301	292	284	276	267	259	242	225	208	191
34 M	10 x "	278	271	263	256	248	241	233	226	211	196	181	166
33 M	9½ x "	258	251	244	237	230	223	216	210	196	182	168	154
32 M	9¼ x "	238	231	225	218	212	205	199	193	180	167	154	141
31 M	9½ x "	217	211	205	199	193	187	181	175	163	151	139	127
30 M	9 x "	196	191	185	179	174	168	163	157	146	135	123	112
28 M	8½ x "	176	171	165	160	155	150	145	140	131	122	112	98.8
22 M	9½ x 12	193	187	180	174	168	162	156	149	137	125	113	100
21 M	9¼ x "	176	170	164	159	153	147	142	136	125	113	102	91.1
20 M	9 x "	159	154	149	143	138	133	128	123	112	102	92.3	82.0
19 M	8½ x "	142	137	133	128	123	119	114	109	100	91.4	82.1	72.8
10 M	9 x 10	111	106	102	97.8	93.5	89.0	84.5	80.0	71.1	62.2	53.3	44.4
9 M	8½ x "	97.7	93.8	89.8	85.5	81.1	77.9	74.0	70.0	62.1	54.2	46.3	38.4

Notes: 1-in. diam. at 6-in. pitch.

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160.

Safe loads are in accordance with the working stresses prescribed by the London County Council (General Powers) Act, 1909, for stanchions of mild steel having "both ends fixed."

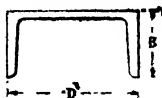
For other conditions and formulae, see notes commencing page 182 L.

Safe loads printed in italics are for heights greater than 40 ft.

For explanations of properties, &c., see Part IV.



# REDPATH, BROWN & CO., LIMITED.



## STANCHIONS.

### Steel Channels.

Safe Concentric Loads, in Tons.  
Ends Fixed.

Reference Mark.	Size, D x B inches.	HEIGHTS IN FEET.												
		2	3	4	5	6	7	8	9	10	11	12	13	
*27 N	15 x 4	73.3	69.9	66.5	63.1	59.7	56.3	52.9	49.5	46.1	42.6	39.2	35.8	
26 N	12 x 4	64.0	61.1	58.3	55.4	52.6	49.7	46.9	44.0	41.1	38.3	35.4	32.6	
*25 N	12 x 3½	56.8	53.7	50.7	47.7	44.7	41.7	38.8	35.7	32.7	29.7	26.6	23.6	
*24 N	12 x 3½	45.2	42.9	40.6	38.3	36.0	33.7	31.4	29.1	26.7	24.3	22.0	19.7	
23 N	11 x 4	58.3	55.8	53.3	50.7	48.1	45.6	43.0	40.5	37.9	35.3	32.7	30.2	
22 N	11 x 3½	51.6	48.9	46.2	43.5	40.8	38.1	35.4	32.7	30.0	27.3	24.6	21.9	
21 N	10 x 4	53.0	50.8	48.5	46.2	43.9	41.6	39.3	37.0	34.7	32.4	30.1	27.8	
*20 N	10 x 3½	48.9	46.4	43.9	41.4	38.9	36.4	33.9	31.4	28.9	26.4	23.9	21.4	
19 N	10 x 3½	40.9	38.8	36.7	34.6	32.5	30.4	28.3	26.2	24.1	22.0	19.9	17.8	
18 N	9 x 4	50.2	48.1	46.0	43.9	41.8	39.7	37.6	35.5	33.4	31.3	29.2	27.1	
*17 N	9 x 3½	44.1	41.9	39.7	37.5	35.3	33.1	30.9	28.7	26.5	24.3	22.1	19.9	
*16 N	9 x 3½	38.7	36.8	34.9	33.0	31.1	29.2	27.3	25.4	23.5	21.6	19.7	17.8	
15 N	9 x 3	32.9	31.2	29.5	27.8	26.1	24.4	22.7	21.0	19.3	17.6	15.9	14.2	
14 N	8 x 4	45.3	43.4	41.5	39.6	37.7	35.8	33.9	32.0	30.1	28.2	26.3	24.4	

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 180.

Safe loads are in accordance with the working stresses prescribed by the London County Council (General Powers) Act, 1900, for stanchions of mild steel having "both ends fixed".

For other conditions and formulae, see notes commencing page 192 L.

Safe loads printed in italics are for heights greater than 40 ft.

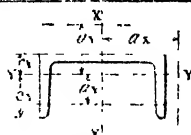
For explanations of properties, etc., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## STANCHIONS.

### Steel Channels.

#### Dimensions and Properties.



Size, D x B inches.	Weight per foot in lbs.	Area in square inches.	Dis- tance e <sub>y</sub> inches	Radii of Gyration.		Eccentricity Coefficients.				
				Axis Y-Y	Axis X-X	Web.	Flange.	Axis Y-Y e <sub>y</sub>	Axis Y-Y e <sub>x</sub>	Axis X-X
*15 x 4	41.94	12.334	3.065	1.08	5.53	1.73	2.84	1 + 2.60e <sub>y</sub>	1 + 0.79e <sub>x</sub>	1 + 0.25e <sub>x</sub>
12 x 4	36.47	10.727	2.969	1.13	4.51	1.83	2.77	1 + 2.34e <sub>y</sub>	1 + 0.81e <sub>x</sub>	1 + 0.30e <sub>x</sub>
*12 x 3½	32.88	9.671	2.633	0.96	4.44	1.82	2.83	1 + 2.86e <sub>y</sub>	1 + 0.94e <sub>x</sub>	1 + 0.31e <sub>x</sub>
*12 x 3¼	26.10	7.675	2.640	0.99	4.54	1.75	2.74	1 + 2.68e <sub>y</sub>	1 + 0.87e <sub>x</sub>	1 + 0.29e <sub>x</sub>
11 x 4	33.22	9.771	2.937	1.14	4.17	1.86	2.74	1 + 2.24e <sub>y</sub>	1 + 0.81e <sub>x</sub>	1 + 0.32e <sub>x</sub>
11 x 3½	29.82	8.771	2.604	0.98	4.11	1.81	2.72	1 + 2.71e <sub>y</sub>	1 + 0.93e <sub>x</sub>	1 + 0.33e <sub>x</sub>
10 x 4	30.16	8.871	2.808	1.16	3.84	1.90	2.70	1 + 2.14e <sub>y</sub>	1 + 0.82e <sub>x</sub>	1 + 0.34e <sub>x</sub>
*10 x 3½	28.21	8.296	2.567	0.99	3.77	1.82	2.76	1 + 2.60e <sub>y</sub>	1 + 0.95e <sub>x</sub>	1 + 0.35e <sub>x</sub>
10 x 3¼	23.55	6.925	2.567	1.02	3.85	1.84	2.69	1 + 2.47e <sub>y</sub>	1 + 0.90e <sub>x</sub>	1 + 0.34e <sub>x</sub>
9 x 4	28.55	8.396	2.849	1.17	3.18	1.96	2.67	1 + 2.06e <sub>y</sub>	1 + 0.83e <sub>x</sub>	1 + 0.37e <sub>x</sub>
*9 x 3½	25.39	7.465	2.529	1.01	3.45	1.92	2.72	1 + 2.47e <sub>y</sub>	1 + 0.95e <sub>x</sub>	1 + 0.38e <sub>x</sub>
*9 x 3¼	22.27	6.550	2.524	1.03	3.49	1.90	2.66	1 + 2.33e <sub>y</sub>	1 + 0.92e <sub>x</sub>	1 + 0.37e <sub>x</sub>
9 x 3	19.37	5.696	2.246	0.84	3.38	1.81	2.77	1 + 3.19e <sub>y</sub>	1 + 1.07e <sub>x</sub>	1 + 0.40e <sub>x</sub>
8 x 4	25.73	7.569	2.790	1.19	3.12	2.01	2.64	1 + 1.97e <sub>y</sub>	1 + 0.84e <sub>x</sub>	1 + 0.41e <sub>x</sub>

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

Each weight per foot is for the shaft only. Weight of case, etc., to be added.

Least radii of gyration and relative eccentricity coefficients are given in parenthesis type.

We = actual eccentric load; K = relative eccentricity coefficient. e<sub>y</sub> = equivalent concentric value; We = We x K.

In axial eccentricity coefficient substitute actual value of "arm of eccentricity" for e<sub>y</sub> and e<sub>x</sub> respectively.

Sections marked (\*) are in our stocks.

For full explanations of tables, see notes commencing page 152 L.

## REDPATH, BROWN &amp; CO., LIMITED.



# STANCHIONS.

## Steel Channels.

Safe Concentric Loads, in Tons.  
Ends Fixed.

Reference Mark.	Size, D x B inches.	HEIGHTS IN FEET.												
		2	3	4	5	6	7	8	9	10	11	12	13	
*13 N	8 x 3½	39.5	37.5	35.35	33.35	31.31	29.27	27.8	25.8	23.9	21.9	20.0	18.1	
12 N	8 x 3	32.9	31.0	29.0	27.1	25.1	23.2	21.2	19.3	17.3	15.4			
11 N	8 x 2½	25.1	23.3	21.4	19.5	17.7	15.8	14.0	12.1					
*10 N	7 x 3½	35.2	33.3	31.3	29.4	27.4	25.5	23.5	21.5	19.5	17.6	15.6	13.6	
9 N	7 x 3	30.0	28.3	26.5	24.7	22.8	21.0	19.1	17.2	15.3	13.4			
8 N	6 x 3½	31.2	29.7	28.1	26.5	24.8	23.2	21.5	19.8	18.1	16.3	14.5		
*7 N	6 x 3	27.9	26.3	24.7	23.0	21.4	19.8	18.1	16.5	14.8	13.1			
*6 N	6 x 3	24.8	23.3	21.7	20.0	18.4	16.7	15.0	13.3	11.6	9.9	8.2	6.5	
5 N	6 x 2½	20.1	18.6	17.1	15.5	13.9	12.3	10.7	9.1	7.4	5.8	4.2	2.6	
*4 N	5 x 2½	18.3	17.0	15.7	14.4	13.1	11.8	10.5	9.2					
*3 N	4 x 2	12.8	11.7	10.5	9.3	8.1	7.0	5.8						
2 N	3½ x 2	10.9	9.9	8.9	7.9	6.9	5.9	4.9						
*1 N	3 x 1½	7.9	6.8	5.8	4.7									

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160

Safe loads are in accordance with the working stresses prescribed by the London County Council (General Powers) Act, 1909, for stanchions of mild steel having "both ends fixed."

For other conditions and formulae, see notes commencing page 192L.

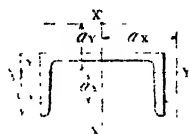
Safe loads printed in italics are for heights greater than 40B

For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## STANCHIONS. Steel Channels.

### Dimensions and Properties.



Size, D x B inches.	Weight per foot in lbs.	Area in square inches.	Dis- tance e <sub>y</sub> inches.	Radii of Gyration.		Eccentricity Coefficients				
				Axis Y-Y	Axis X-X	Web.	Flange.	Axis Y-Y e <sub>y</sub>	Axis Y-Y e <sub>y</sub>	Axis X-X
*8 x 3½	22.72	6.682	2.489	1.03	3.09	1.97	2.68	1+2.36a <sub>y</sub>	1+0.96a <sub>y</sub>	1+0.42a <sub>x</sub>
8 x 3	19.30	5.675	2.156	0.87	3.07	1.94	2.70	1+2.83a <sub>y</sub>	1+1.11a <sub>y</sub>	1+0.43a <sub>x</sub>
8 x 2½	15.12	4.448	1.834	0.71	3.04	1.87	2.73	1+3.58a <sub>y</sub>	1+1.30a <sub>y</sub>	1+0.43a <sub>x</sub>
*7 x 3½	20.23	5.950	2.439	1.04	2.73	2.13	2.64	1+2.24a <sub>y</sub>	1+0.97a <sub>y</sub>	1+0.47a <sub>x</sub>
7 x 3	17.56	5.166	2.126	0.88	2.70	1.98	2.65	1+2.74a <sub>y</sub>	1+1.13a <sub>y</sub>	1+0.48a <sub>x</sub>
6 x 3½	17.90	5.266	2.381	1.06	2.37	2.12	2.66	1+2.12a <sub>y</sub>	1+1.00a <sub>y</sub>	1+0.53a <sub>x</sub>
*6 x 3	16.29	4.791	2.072	0.89	2.33	2.08	2.66	1+2.60a <sub>y</sub>	1+1.17a <sub>y</sub>	1+0.55a <sub>x</sub>
*6 x 3	14.49	4.261	2.062	0.90	2.37	2.07	2.60	1+2.51a <sub>y</sub>	1+1.14a <sub>y</sub>	1+0.53a <sub>x</sub>
6 x 2½	12.04	3.542	1.796	0.73	2.30	1.93	2.70	1+3.88a <sub>y</sub>	1+1.83a <sub>y</sub>	1+0.57a <sub>x</sub>
*5 x 2½	10.98	3.230	1.743	0.74	1.94	2.05	2.67	1+3.18a <sub>y</sub>	1+1.38a <sub>y</sub>	1+0.67a <sub>x</sub>
*4 x 2	7.96	2.341	1.344	0.60	1.56	2.20	2.64	1+3.74a <sub>y</sub>	1+1.83a <sub>y</sub>	1+0.82a <sub>x</sub>
3½ x 2	6.75	1.986	1.355	0.80	1.36	2.16	2.65	1+3.78a <sub>y</sub>	1+1.80a <sub>y</sub>	1+0.94a <sub>x</sub>
*3 x 1½	5.27	1.549	1.016	0.43	1.13	2.23	2.68	1+5.32a <sub>y</sub>	1+2.54a <sub>y</sub>	1+1.12a <sub>x</sub>

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

Each weight per foot is for the shaft only. Weight of base, &c., to be added.

Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

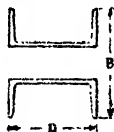
We = actual eccentric load; K = relative eccentricity coefficient; We = equivalent concentric value; We = We x K.

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for a<sub>y</sub> and a<sub>x</sub> respectively.

Sections marked (\*) are in our stocks.

For full explanations of tables, see notes commencing page 192 L.

REDPATH, BROWN & CO., LIMITED.



COMPOUND STANCHIONS.

Safe Concentric Loads, in Tons.  
Ends Fixed.

Reference Mark.	Size, D x B inches	HEIGHTS IN FEET.											
		10	12	14	16	18	20	22	24	28	32	36	40
25 O	12 x 13	111	108	105	102	99	596	593	690	784	979	173	267
20 O	10 x 12	93	90	88	85	82	779	777	274	468	863	257	752
17 O	9 x 11	82	80	77	74	71	468	465	662	857	151	445	739
13 O	8 x 10	72	69	66	63	60	357	354	351	345	439	535	627
10 O	7 x 9	61	58	55	52	49	246	243	240	234	277	321	375

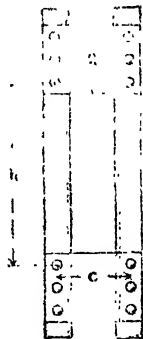


SINGLE LATTICING

Suitable for values of  $\lambda$  not exceeding 110.



DOUBLE LATTICING



BATTEN PLATES.

The above safe loads are tabulated for ratio of slenderness up to, but not exceeding 166.  
Safe loads are in accordance with the working stresses prescribed by the London County Council (General Powers) Act, 1903, for stanchions of mild steel having a both ends fixed.  
For other conditions and formulae, see notes commencing page 192 L.  
Safe loads printed in italics are for heights greater than 40 ft.  
For explanations of properties, &c., see Part IV.

### COMPOUND STANCHIONS.

### Composition and Properties.

Composed of Two Steel Channels Laced.	Weight per foot in lbs.	Area in square inches	1 Space between Webs, Inches	Radius of Gyration		Eccentricity Coefficients			
				Axis Y-Y	Axis X-X	Web.	Flange.	Axis Y-Y	Axis X-X
12 x 3½	66	19.3	6	3.98	4.14	3.15	2.83	1.0417	1.0317
10 x 3½	58½	16.6	5	3.87	3.77	3.22	2.76	1.0477	1.0357
9 x 3½	51	14.9	4	3.14	3.43	3.35	2.72	1.0567	1.0387
8 x 3½	45½	13.3	3	2.71	3.09	3.59	2.68	1.0687	1.0427
7 x 3½	40½	11.9	2	2.31	2.73	3.66	2.64	1.0847	1.0477

CONVENTIONAL MAXIMUM SPACING AND MINIMUM PROPORTIONS OF LATTICE BARS AND  
BATTEN PLATES FOR CONCRETE UNDER (1)  $f_p$ ,  $f_c$ ,  $f_y$ ,  $f_r$ , and Maintenance of  
Flow Assumptions

Depth of Channel, Inches.	12	14	9	8	7
Width of Lattice Bar, Inches.	2½	2½	2½	2½	2
Diameter of Rivet.	¾	¾	¾	¾	¾

Maximum angle of inclination with horizontal = 30 degrees  
Minimum thickness = 1/40th of  $\lambda$ , the wavelength of the light  
Maximum horizontal centres of rivets,  $c = 5$  inches

Minimum thickness =  $1/600h$  of  $a$ , the diagonal centre of rivets

Let  $t$  = height of stanchion in inches, and  $k$  = radius of gyration of one channel.  
Then  $k \geq \frac{t \times k_{\text{least}}}{k_{\text{greatest}}}$ .

Minimum thickness =  $1/50$ th of  $c$ , the horizontal centres of rivets.

Minimum width  $g = c$ , the horizontal centres of rivets for end plates.

" "  $\theta = 45^\circ$ , " " " intermediate plates.

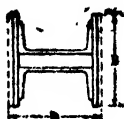
In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 14% per cent over this must be allowed. See page 7.

Each weight per foot is for the shaft only. Weights of lattices, base, &c., to be added.  
Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

We = axial eccentricity loci,  $k$  = relative eccentricity coefficient;  $W_e$  = equivalent eccentricity value,  $W_e = W_e / K$ . In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for  $\alpha_x$  and  $\alpha_z$  respectively.

For full explanations of tables, see notes commencing page 192.

## REDPATH, BROWN &amp; CO., LIMITED.



## COMPOUND STANCHIONS.

Safe Concentric Loads, in Tons.  
Ends Fixed.

Reference Mark	Size, D x B inches.	HEIGHTS IN FEET.												
		10	11	12	13	14	16	18	20	24	28	32	36	
58 P	15 x 14	348	343	338	333	328	318	309	299	279	259	239	219	
56 P	14½ x "	308	304	299	295	290	281	272	263	245	227	210	192	
54 P	14 x "	268	264	260	256	252	244	236	228	212	196	180	165	
53 P	13½ x "	247	244	240	236	232	225	218	210	195	181	166	151	
52 P	13¼ x "	227	224	220	217	213	206	199	192	179	165	151	137	
51 P	13½ x "	207	204	200	197	194	188	181	175	162	149	136	123	
50 P	13 x "	187	184	181	178	175	169	163	157	145	133	121	109	
49 P	12¾ x 12	153	150	146	143	140	134	128	122	109	97-2	84-8	72-4	
38 P	13 x 12	292	287	282	277	272	262	252	242	222	202	182	162	
36 P	12½ x "	258	253	249	244	240	231	222	213	195	177	159	141	
34 P	12 x "	224	220	216	212	208	200	192	184	168	152	137	121	
33 P	11½ x "	207	203	199	196	192	185	177	170	155	140	125	110	
32 P	11½ x "	190	186	183	179	176	169	162	155	141	128	114	100	
31 P	11¼ x "	173	170	166	163	160	154	147	141	128	115	102	89-8	
30 P	11 x "	156	153	150	147	144	138	132	126	114	102	90-9	79-1	
29 P	10½ x 10	124	121	118	115	112	106	99-7	93-3	80-7	68-1			
Rivets ½-in. diam. at 6-in. pitch.														

Rivets ½-in. diam. at 6-in. pitch.

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160.

Safe loads are in accordance with the working stresses prescribed by the London County Council (General Powers) Act, 1909, for stanchions of mild steel having "both ends fixed."

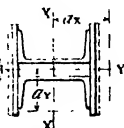
For other conditions and formulae, see notes commencing page 192 L.

For explanations of properties, etc., see Part IV.

## REDPATH, BROWN &amp; CO., LIMITED.

## COMPOUND STANCHIONS.

## Composition and Properties.



Composed of		Weight per foot in lbs.	Area in square inches	d Space between Webs in inches.	Radii of Gyration.		Eccentricity Coefficients.			
Two Steel Channels.	Plates, each flange to form				Axis Y-Y	Axis X-X	Web.	Flange.	Axis Y-Y	Axis X-X
12 x 3½	14 x 1½	211	61.3	3.5	3.60	6.12	2.16	2.50	1+0.52a <sub>v</sub>	1+0.20a <sub>x</sub>
"	" x 1½	187½	54.3	"	3.64	5.91	2.19	2.49	1+0.53a <sub>v</sub>	1+0.21a <sub>x</sub>
"	" x 1	163½	47.3	"	3.58	5.75	2.23	2.48	1+0.55a <sub>v</sub>	1+0.21a <sub>x</sub>
"	" x ¾	150	43.8	"	3.54	5.64	2.26	2.48	1+0.56a <sub>v</sub>	1+0.22a <sub>x</sub>
"	" x ¾	139½	40.3	"	3.49	5.53	2.29	2.49	1+0.57a <sub>v</sub>	1+0.22a <sub>x</sub>
"	" x ¾	129	36.8	"	3.44	5.41	2.33	2.50	1+0.59a <sub>v</sub>	1+0.23a <sub>x</sub>
"	" x ½	116	33.3	"	3.37	5.28	2.39	2.52	1+0.62a <sub>v</sub>	1+0.24a <sub>x</sub>
"	12 x ¾	99	28.5	2.5	2.74	5.06	2.40	2.59	1+0.80a <sub>v</sub>	1+0.25a <sub>x</sub>
10 x 3½	12 x 1½	181½	52.6	2.5	3.16	5.29	2.03	2.51	1+0.60a <sub>v</sub>	1+0.23a <sub>x</sub>
"	" x 1½	161	46.6	"	3.12	5.05	2.06	2.53	1+0.62a <sub>v</sub>	1+0.25a <sub>x</sub>
"	" x 1	140½	40.6	"	3.07	4.87	2.10	2.52	1+0.64a <sub>v</sub>	1+0.25a <sub>x</sub>
"	" x ¾	130½	37.6	"	3.04	4.78	2.12	2.51	1+0.65a <sub>v</sub>	1+0.26a <sub>x</sub>
"	" x ¾	120	34.6	"	3.00	4.68	2.15	2.51	1+0.67a <sub>v</sub>	1+0.26a <sub>x</sub>
"	" x ¾	110	31.6	"	2.95	4.57	2.19	2.52	1+0.69a <sub>v</sub>	1+0.27a <sub>x</sub>
"	" x ½	99½	28.6	"	2.89	4.46	2.2½	2.53	1+0.72a <sub>v</sub>	1+0.28a <sub>x</sub>
"	10 x ¾	84½	24.1	1.5	2.28	4.26	2.17	2.59	1+0.96a <sub>v</sub>	1+0.30a <sub>x</sub>

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

Each weight per foot is for the riveted section only. Weight of base, &c., to be added.

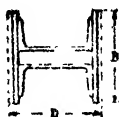
Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

We=actual eccentric load; K=relative eccentricity coefficient; Wc=equivalent concentric value; Wc=WxK.

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for a, and a<sub>v</sub> respectively.

For full explanations of tables, see notes commencing page 192L.

REDPATH, BROWN & CO., LIMITED.



COMPOUND STANCHIONS.

Safe Concentric Loads, in Tons.  
Ends Fixed.

Reference Mark	Size D x B inches	HEIGHTS IN FEET.											
		10	11	12	13	14	16	18	20	24	28	32	36
24 P	11 x 12	215	213	207	204	200	192	185	177	162	147	132	117
23 P	10 1/2 x "	198	195	191	188	184	177	170	163	149	135	121	107
22 P	10 1/4 x "	181	178	175	171	168	162	155	149	136	123	110	97 1/2
21 P	10 1/2 x "	164	161	158	155	152	146	140	134	122	110	98 7/8	86 7/8
20 P	10 x "	147	144	142	139	137	131	125	120	109	98 1/8	87 1/2	76 1/2
19 P	9 3/4 x 10	116	114	111	108	105	99	93	88	76	65	55	45
14 P	10 x 10	178	174	170	166	162	154	147	139	123	108	92	81
13 P	9 3/4 x "	164	160	157	153	149	142	135	128	113	99	84	76
12 P	9 1/2 x "	150	147	143	140	137	130	123	116	103	89	76	64
11 P	9 1/4 x "	136	133	130	127	124	118	111	105	93	80	68	58
10 P	9 x "	123	120	117	114	111	105	100	94	82	71	59	49
9 P	8 3/4 x 9	102	99	96	93	90	85	79	73	62	50	40	30
Rivets 1/4-in diam. at 6-in pitch.													

Rivets 1/2-in diam. at 6-in pitch.

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160.

Safe loads are in accordance with the working stresses prescribed by the London County Council (General Powers) Act, 1900, for stanchions of mild steel having "both ends fixed."

For other conditions and formulae, see notes commencing page 192 L.

Safe loads printed in italics are for heights greater than 40 ft.

For explanations of properties, &c., see Part IV.

## REDPATH, BROWN &amp; CO., LIMITED.

## COMPOUND STANCHIONS.

## Composition and Properties.

T. & S. Steel Bands	T. & S. Steel Bands	T. & S. Steel Bands	T. & S. Steel Bands	T. & S. Steel Bands	T. & S. Steel Bands	T. & S. Steel Bands	T. & S. Steel Bands	T. & S. Steel Bands	Eccentricity Coefficients		
									Axis Y-Y	Axis X-X	
8 x 3 1/2	12 x 1	130	158	2 1/2	6 1/2	1 1/2	2 1/2	2 1/2	1+0.62a	1+0.28a	
"	" x 1 1/2	124 1/2	155 1/2	"	6 1/2	1 1/2	2 1/2	2 1/2	1+0.63a	1+0.28a	
"	" x 2	114 1/2	141 1/2	"	6 1/2	1 1/2	2 1/2	2 1/2	1+0.65a	1+0.29a	
"	" x 2 1/2	104 1/2	127 1/2	"	6 1/2	1 1/2	2 1/2	2 1/2	1+0.67a	1+0.30a	
"	" x 3	94	113	"	6 1/2	1 1/2	2 1/2	2 1/2	1+0.70a	1+0.30a	
"	" x 3 1/2	79	97 1/2	"	6 1/2	1 1/2	2 1/2	2 1/2	1+0.92a	1+0.32a	
8 x 3 1/2	10 x 1	110	138	2 1/2	6 1/2	1 1/2	2 1/2	2 1/2	1+0.75a	1+0.31a	
"	" x 1 1/2	104 1/2	131 1/2	"	6 1/2	1 1/2	2 1/2	2 1/2	1+0.77a	1+0.32a	
"	" x 2	94	121 1/2	"	6 1/2	1 1/2	2 1/2	2 1/2	1+0.79a	1+0.33a	
"	" x 2 1/2	84 1/2	107 1/2	"	6 1/2	1 1/2	2 1/2	2 1/2	1+0.81a	1+0.33a	
"	" x 3	74 1/2	93 1/2	"	6 1/2	1 1/2	2 1/2	2 1/2	1+0.84a	1+0.34a	
"	" x 3 1/2	64 1/2	81 1/2	"	6 1/2	1 1/2	2 1/2	2 1/2	1+1.01a	1+0.36a	

In each case the weight per foot given is the minimum that can be rolled, and a rolling

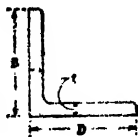
gain of 1 1/2 per cent. over this is to be allowed. See page 7.

Each weight per foot is for the riveted joint only. Weight of base, &amp;c., to be added.

Least radii of gyration and relative eccentricity coefficients are printed in prominent type. The radii of gyration and the relative eccentricity coefficient  $W_{ec}$  equivalent concentricaxis,  $W_{ec}$  =  $W_{ec}$ .In axial eccentricity coefficients substitute actual value of "sum of eccentricity" for  $a$  and  $a_x$  respectively.

For full explanations of tables see notes commencing page 192 L.

## REDPATH, BROWN &amp; CO., LIMITED.



# STANCHIONS (or STRUTS).

## Steel Equal Angles.

Safe Concentric Loads, in Tons.  
Ends Fixed.

Reference Mark.	Size D x B x t inches	HEIGHTS IN FEET.													
		2	3	4	5	6	7	8	9	10	11	12	14	16	18
14g Q	6 x 6 x $\frac{3}{8}$	50.5	48.3	46.2	44.0	41.8	39.7	37.5	35.3	33.2	31.0	28.9	24.6		
14f Q	" x $\frac{3}{8}$	42.6	40.8	39.0	37.2	35.3	33.5	31.7	29.9	28.1	26.3	24.5	20.9		
14e Q	" x $\frac{1}{2}$	34.4	33.0	31.5	30.0	28.5	27.1	25.7	24.2	22.7	21.3	19.8	16.4		
13g Q	5 x 5 x $\frac{3}{8}$	40.7	38.5	36.4	34.2	32.0	29.9	27.7	25.5	23.4	21.2	19.0			
13f Q	" x $\frac{3}{8}$	34.4	32.7	31.0	29.2	27.3	25.5	23.7	21.9	20.1	18.3	16.5			
13e Q	" x $\frac{1}{2}$	27.9	26.5	25.0	23.6	22.1	20.7	19.2	17.7	16.3	14.8	13.4			
12g Q	4½ x 4½ x $\frac{3}{8}$	35.8	33.6	31.4	29.2	27.0	24.9	22.7	20.5	18.3	16.2				
12f Q	" x $\frac{3}{8}$	30.4	28.6	26.8	25.0	23.2	21.4	19.5	17.7	15.9	14.1				
12e Q	" x $\frac{1}{2}$	24.7	23.2	21.7	20.2	18.7	17.3	15.9	14.4	12.9	11.5				
11g Q	4 x 4 x $\frac{3}{8}$	31.0	28.8	26.6	24.4	22.2	20.0	17.8	15.6	13.5					
11f Q	" x $\frac{3}{8}$	26.3	24.5	22.7	20.9	19.1	17.3	15.5	13.7	12.0					
11e Q	" x $\frac{1}{2}$	21.4	19.9	18.5	17.0	15.6	14.1	12.6	11.2	9.7					
11d Q	" x $\frac{3}{8}$	16.3	15.2	14.1	13.0	11.9	10.8	9.7	8.6	7.5					
10f Q	3½ x 3½ x $\frac{3}{8}$	22.4	20.6	18.9	17.1	15.4	13.6	11.8	10.1						
10e Q	" x $\frac{1}{2}$	18.2	16.8	15.4	13.9	12.5	11.0	9.6	8.2						
10d Q	" x $\frac{3}{8}$	13.9	12.8	11.7	10.6	9.5	8.4	7.3	6.2						

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 100.

Safe loads are in accordance with the working stresses prescribed by the London County Council (General Powers) Act, 1909, for stanchions of mild steel having "both ends fixed."

For other conditions and formulae, see notes commencing page 192 L.

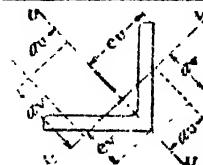
For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## STANCHIONS (or STRUTS).

### Steel Equal Angles.

#### Dimensions and Properties.



Size, D x B x t inches.	Weight per foot in lbs.	Area in square inches	Distances in inches		Radii of Gyration		Eccentricity Coefficients.	
			$e_h$	$e_v$	Axis V-V	Axis U-U	Axis V-V	Axis U-U
6 x 6 x $\frac{3}{8}$	28.70	8.441	2.49	4.24	1.17	2.28	1+1.82 <i>a</i>	1+0.82 <i>a</i>
" x 6 x $\frac{5}{8}$	24.13	7.113	2.42	4.21	1.18	2.30	1+1.74 <i>a</i>	1+0.81 <i>a</i>
" x $\frac{3}{4}$	19.55	5.753	2.35	4.24	1.12	2.32	1+1.69 <i>a</i>	1+0.79 <i>a</i>
5 x 5 x $\frac{3}{8}$	23.59	6.938	2.14	3.34	0.96	1.98	1+2.32 <i>a</i>	1+1.00 <i>a</i>
" x 5 x $\frac{5}{8}$	19.92	5.869	2.07	3.34	0.93	1.89	1+2.16 <i>a</i>	1+0.98 <i>a</i>
" x $\frac{3}{4}$	16.15	4.751	2.00	3.34	0.98	1.92	1+2.09 <i>a</i>	1+0.96 <i>a</i>
$4\frac{1}{2}$ x $4\frac{1}{2}$ x $\frac{3}{8}$	21.05	5.189	1.96	3.18	0.95	1.69	1+2.72 <i>a</i>	1+1.12 <i>a</i>
" x $\frac{5}{8}$	17.10	4.236	1.90	3.15	0.87	1.70	1+2.51 <i>a</i>	1+1.10 <i>a</i>
" x $\frac{3}{4}$	14.46	4.252	1.83	3.18	0.87	1.72	1+2.42 <i>a</i>	1+1.07 <i>a</i>
4 x 4 x $\frac{3}{8}$	18.43	5.437	1.79	2.87	0.76	1.43	1+3.10 <i>a</i>	1+1.28 <i>a</i>
" x $\frac{5}{8}$	15.66	4.609	1.72	2.83	0.77	1.50	1+2.91 <i>a</i>	1+1.26 <i>a</i>
" x $\frac{3}{4}$	12.75	3.759	1.56	2.83	0.77	1.52	1+2.80 <i>a</i>	1+1.22 <i>a</i>
" x $\frac{5}{8}$	9.72	2.859	1.59	2.83	0.78	1.54	1+2.61 <i>a</i>	1+1.19 <i>a</i>
$3\frac{1}{2}$ x $3\frac{1}{2}$ x $\frac{3}{8}$	13.55	3.985	1.55	2.47	0.68	1.29	1+3.35 <i>a</i>	1+1.47 <i>a</i>
" x $\frac{5}{8}$	11.05	3.251	1.48	2.47	0.68	1.32	1+3.21 <i>a</i>	1+1.43 <i>a</i>
" x $\frac{3}{4}$	8.45	2.485	1.41	2.47	0.68	1.34	1+3.06 <i>a</i>	1+1.38 <i>a</i>

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

Each weight per foot is for the shaft only. Weight of connections, &c., to be added.

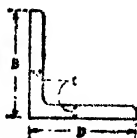
Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

W=actual eccentric load; K=relative eccentricity coefficient; Wc=equivalent concentric value; Wc=WxK.

In axial eccentricity coefficients substitute actual value of "axis of eccentricity" for *a* and *a* respectively.

For full explanations of tables, see notes commencing page 182 L.

REDPATH, BROWN & CO., LIMITED.



**STANCHIONS (or STRUTS).**  
**Steel Equal Angles.**

Safe Concentric Loads, in Tons.  
Ends Fixed.

Reference Mark.	Size, in inches	HEIGHTS IN FEET.						
		2	3	4	5	6	7	
9y Q	3 x 3 x $\frac{3}{8}$	18.3	16.1	14.0	13.1	11.4	9.6	—
9x Q	" x $\frac{3}{8}$	15.0	13.0	12.2	10.7	9.3	7.9	—
9z Q	" x $\frac{3}{8}$	11.5	10.4	9.2	8.2	7.1	6.0	—
9c Q	" x $\frac{3}{8}$	9.7	8.7	7.8	6.9	6.0	5.1	—
9b Q	" x $\frac{3}{8}$	7.8	7.1	6.4	5.7	4.9	4.2	—
7e Q	2½ x 2½ x $\frac{3}{8}$	11.8	10.4	9.0	7.7	6.1		—
7d Q	" x $\frac{3}{8}$	9.1	8.0	6.9	5.8	4.7		—
7c Q	" x $\frac{3}{8}$	7.6	6.7	5.8	4.9	4.0		—
7b Q	" x $\frac{3}{8}$	6.2	5.3	4.5	3.7	3.2		—
6c Q	2½ x 2½ x $\frac{3}{8}$	6.6	5.7	4.8	3.9			—
6b Q	" x $\frac{3}{8}$	5.4	4.5	3.9	3.1			—
5b Q	2 x 2 x $\frac{3}{8}$	4.6	3.9	3.2	2.4			—
5a Q	" x $\frac{3}{8}$	3.5	3.0	2.4	1.9			—

The above safe loads are calculated for ratios of slenderness up to, but not exceeding 160.

Side loads are in accordance with the working stresses prescribed by the London County Council (General Powers) Act, 1900, for stanchions of mild steel having "both ends fixed."

For other conditions and formulae, see notes commencing page 192 L.

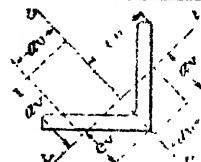
For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## STANCHIONS (or STRUTS).

### Steel Equal Angles.

#### Dimensions and Properties.



Size, D x B x t inches.	Weight per foot in lbs.	Area in square inches.	Distances in inches		Radii of Gyration.		Eccentricity Coefficients.	
			$x_v$	$x_u$	Axis V-V	Axis U-U	Axis V-V	Axis U-U
3 x 3 x $\frac{1}{2}$	11.43	3.362	1.37	2.12	0.58	1.09	1 + 4.07 $a_v$	1 + 1.76 $a_u$
" x $\frac{1}{2}$	9.36	2.753	1.31	2.12	0.58	1.12	1 + 3.89 $a_v$	1 + 1.70 $a_u$
" x $\frac{3}{4}$	7.18	2.112	1.24	2.12	0.58	1.13	1 + 3.69 $a_v$	1 + 1.64 $a_u$
" x $\frac{7}{8}$	6.05	1.776	1.21	2.12	0.58	1.15	1 + 3.60 $a_v$	1 + 1.61 $a_u$
" x $\frac{1}{2}$	4.90	1.440	1.17	2.12	0.59	1.15	1 + 3.37 $a_v$	1 + 1.60 $a_u$
2 $\frac{1}{2}$ x 2 $\frac{1}{2}$ x $\frac{1}{2}$	7.65	2.250	1.13	1.77	0.48	0.91	1 + 4.90 $a_v$	1 + 2.12 $a_u$
" x $\frac{3}{4}$	5.89	1.734	1.06	1.77	0.48	0.93	1 + 4.62 $a_v$	1 + 2.02 $a_u$
" x $\frac{7}{8}$	4.96	1.460	1.03	1.77	0.48	0.94	1 + 4.49 $a_v$	1 + 1.97 $a_u$
" x $\frac{1}{2}$	4.04	1.187	0.99	1.77	0.48	0.95	1 + 4.32 $a_v$	1 + 1.91 $a_u$
2 $\frac{1}{2}$ x 2 $\frac{1}{2}$ x $\frac{3}{4}$	4.45	1.310	0.94	1.59	0.43	0.84	1 + 5.11 $a_v$	1 + 2.22 $a_u$
" x $\frac{1}{2}$	3.61	1.061	0.91	1.59	0.44	0.85	1 + 4.72 $a_v$	1 + 2.20 $a_u$
2 x 2 x $\frac{1}{2}$	3.19	0.940	0.82	1.41	0.39	0.74	1 + 5.39 $a_v$	1 + 2.56 $a_u$
" x $\frac{3}{4}$	2.43	0.720	0.78	1.41	0.30	0.75	1 + 5.12 $a_v$	1 + 2.49 $a_u$

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

Each weight per foot is for the shaft only. Weight of connections, &c., to be added.

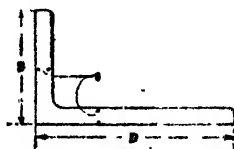
Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

W = actual eccentric load; K = relative eccentricity coefficient; Wc = equivalent concentric value; Wc = W x K.

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for  $a_v$  and  $a_u$  respectively.

For full explanations of tables, see notes commencing page 192 L.

REDPATH, BROWN & CO., LIMITED.



**STANCHIONS (or STRUTS).**  
**Steel Unequal Angles.**

Safe Concentric Loads, in Tons.  
Ends Fixed.

Reference Mark.	Size, D x B x t inches.	HEIGHTS IN FEET.									
		2	3	4	5	6	7	8	9	10	11
25g R	7 x 3½ x ½	41.5	38.5	35.5	32.5	29.5	26.5	23.5	20.4		
25f R	" x ½	35.1	32.6	30.1	27.6	25.1	22.6	20.1	17.6		
25e R	" x ½	28.4	26.1	24.3	22.3	20.3	18.3	16.2	14.2		
21f R	6 x 4 x ½	34.0	31.9	29.9	27.8	25.8	23.7	21.7	19.6	17.6	15.6
21e R	" x ½	27.5	25.9	24.2	22.5	20.9	19.2	17.6	15.9	14.3	12.6
20f R	6 x 3½ x ½	31.6	29.4	27.1	24.9	22.7	20.5	18.3	16.0	13.8	
20e R	" x ½	25.6	23.8	22.0	20.2	18.4	16.6	14.8	13.0	11.2	
20d R	" x ½	19.5	18.2	16.8	15.5	14.1	12.8	11.4	10.0	8.7	
63f R	6 x 3 x ½	29.0	26.5	24.0	21.5	19.0	16.5	14.0			
63e R	" x ½	23.5	21.5	19.5	17.5	15.1	13.4	11.4			
63d R	" x ½	18.0	16.5	14.9	13.4	11.9	10.4	8.9			

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160.

Safe loads are in accordance with the working stresses prescribed by the London County Council (General Powers) Act, 1892, for stanchions of mild steel having "both ends fixed."

For other conditions and formulae, see notes commencing page 192 L.

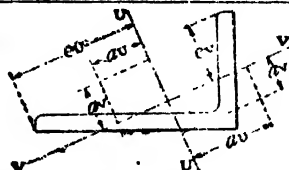
For explanations of properties, &c., see Part IV

REDPATH, BROWN & CO., LIMITED.

STANCHIONS (or STRUTS).

Steel Unequal Angles.

Dimensions and Properties.



Size, D x B x t inches.	Weight per foot in lbs.	Area in square inches.	Distances in inches.		Radii of Gyration.		Eccentricity Coefficients.	
			$e_v$	$e_u$	Axis V-V	Axis U-U	Axis V-V	Axis U-U
$7 \times 3\frac{1}{2} \times \frac{3}{8}$	24.86	7.313	2.03	4.48	0.73	2.27	$1+3.81a_v$	$1+0.87a_u$
" $\times \frac{3}{8}$	20.98	6.172	2.05	4.51	0.74	2.29	$1+3.75a_v$	$1+0.86a_u$
" $\times \frac{1}{2}$	17.00	5.000	2.07	4.55	0.74	2.31	$1+3.78a_v$	$1+0.86a_u$
$6 \times 4 \times \frac{3}{8}$	19.92	5.860	2.08	4.06	0.86	2.01	$1+2.82a_v$	$1+1.00a_u$
" $\times \frac{1}{2}$	16.15	4.750	2.08	4.09	0.86	2.03	$1+2.82a_v$	$1+0.99a_u$
$6 \times 3\frac{1}{2} \times \frac{3}{8}$	18.87	5.550	1.94	3.96	0.74	1.98	$1+3.49a_v$	$1+1.01a_u$
" $\times \frac{1}{2}$	15.31	4.502	1.92	3.99	0.75	2.00	$1+3.42a_v$	$1+1.00a_u$
" $\times \frac{3}{8}$	11.64	3.424	1.96	4.02	0.76	2.01	$1+3.89a_v$	$1+1.00a_u$
$6 \times 3 \times \frac{3}{8}$	17.90	5.236	1.76	3.84	0.63	1.94	$1+4.43a_v$	$1+1.02a_u$
" $\times \frac{1}{2}$	14.46	4.252	1.76	3.88	0.63	1.96	$1+4.46a_v$	$1+1.01a_u$
" $\times \frac{3}{8}$	11.00	3.236	1.80	3.91	0.64	1.98	$1+4.38a_v$	$1+1.00a_u$

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of  $\frac{3}{4}$  per cent. over this must be allowed. See page 7.

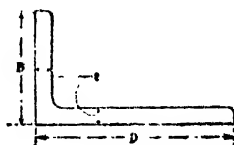
Each weight per foot is for the shaft only. Weight of connections, &c., to be added.

Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

We=actual eccentric load; K=relative eccentricity coefficient; We=equivalent concentric value;  $W_e = W \times K$ .  
In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for  $a_v$  and  $a_u$  respectively.

For full explanations of tables see notes commencing page 192 L.

REDPATH, BROWN & CO., LIMITED.



STANCHIONS (or STRUTS).

Steel Unequal Angles.

Safe Concentric Loads, in Tons.  
Ends Fixed.

Reference Mark.	Size, D x B x t inches.	HEIGHTS IN FEET.									
		2	3	4	5	6	7	8	9	10	11
17f R	5 x 4 x 3/8	30.2	28.3	26.4	24.5	22.6	20.7	18.9	17.0	15.1	13.2
17e R	" x 1/2	24.6	23.0	21.5	20.0	18.5	17.0	15.4	13.9	12.4	10.9
17d R	" x 5/8	18.7	17.6	16.1	15.3	14.1	13.0	11.9	10.7	9.6	8.4
15f R	5 x 3 x 3/8	25.6	23.4	21.3	19.1	17.0	14.8	12.6			
15e R	" x 1/2	20.8	19.1	17.3	15.5	13.8	12.0	10.3			
15d R	" x 5/8	15.9	14.6	13.3	11.9	10.6	9.3	8.0			
11e R	4 x 3 x 1/2	18.0	16.4	14.9	13.3	11.8	10.2	8.7			
11d R	" x 5/8	13.8	12.6	11.4	10.3	9.1	8.0	6.8			
7d R	3 x 2 1/2 x 5/8	10.2	9.1	8.0	6.9	5.8					
7c R	" x 3/4	8.6	7.7	6.7	5.8	4.9					

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160.  
Safe loads are in accordance with the working stresses prescribed by the London County Council (General Powers) Act, 1909, for stanchions of mild steel having "both ends fixed."

For other conditions and formulae, see notes commencing page 192 L.

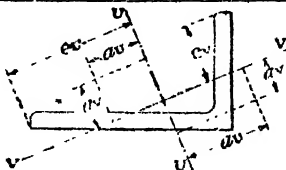
For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## STANCHIONS (or STRUTS).

### Steel Unequal Angles.

#### Dimensions and Properties.



Size, D x B x t inches.	Weight per foot in lbs.	Area in square inches	Distances in inches.		Radii of Gyration.		Eccentricity Coefficients.	
			$e_v$	$e_u$	Axis V-V	Axis U-U	Axis V-V	Axis U-U
5 x 4 x $\frac{3}{8}$	17.80	5.236	1.81	3.46	0.83	1.74	1+2.63 $a_v$	1+1.15 $a_u$
" x $\frac{1}{2}$	14.46	4.252	1.83	3.48	0.84	1.75	1+2.60 $a_v$	1+1.13 $a_u$
" x $\frac{5}{8}$	11.00	3.236	1.82	3.49	0.85	1.77	1+2.52 $a_v$	1+1.12 $a_u$
5 x 3 x $\frac{3}{8}$	15.67	4.609	1.65	3.30	0.64	1.64	1+4.05 $a_v$	1+1.23 $a_u$
" x $\frac{1}{2}$	12.75	3.749	1.65	3.32	0.64	1.66	1+4.02 $a_v$	1+1.21 $a_u$
" x $\frac{5}{8}$	9.72	2.859	1.67	3.36	0.65	1.67	1+3.95 $a_v$	1+1.20 $a_u$
4 x 3 x $\frac{1}{2}$	11.05	3.251	1.45	2.75	0.63	1.36	1+3.66 $a_v$	1+1.48 $a_u$
" x $\frac{3}{4}$	8.45	2.485	1.45	2.77	0.64	1.38	1+3.54 $a_v$	1+1.45 $a_u$
3 x 2 $\frac{1}{2}$ x $\frac{3}{8}$	6.53	1.921	1.11	2.09	0.52	1.05	1+4.10 $a_v$	1+1.90 $a_u$
" x $\frac{1}{2}$	5.51	1.620	1.10	2.10	0.52	1.06	1+4.05 $a_v$	1+1.87 $a_u$

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

Each weight per foot is for the shaft only. Weight of connections, &c., to be added.

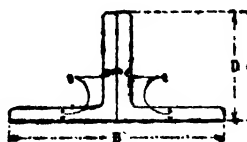
Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

$W_e$  = actual eccentric load;  $K$  = relative eccentricity coefficient;  $W_c$  = equivalent concentric value;  $W_c = W_e \times K$ .

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for  $a$  and  $a_u$  respectively.

For full explanations of tables, see notes commencing page 192 L.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND STANCHIONS (or STRUTS).

Two Steel Equal Angles Back to Back.

Safe Concentric Loads, in Tons.  
Ends Fixed.

Reference Mark.	Size, D x B inches	HEIGHTS IN FEET.															
		2	3	4	5	6	7	8	9	10	12	14	16	18	20	22	24
14g S	6 x 12	104	101	98	95	93	90	87	84	81	76	70	66	65	64	63	62
14f S	"	87	85	82	80	78	76	73	71	69	64	59	55	54	53	52	51
14e S	"	71	69	67	65	63	61	59	57	55	50	45	41	40	39	38	37
13g S	5 x 10	84	81	79	77	75	73	70	68	65	61	56	52	51	50	49	48
13f S	"	71	69	66	64	62	60	57	55	53	48	43	39	38	37	36	35
13e S	"	58	56	54	52	50	48	46	44	42	39	35	31	30	29	28	27
12g S	4 1/2 x 9	75	72	70	68	66	63	61	58	56	52	47	42	41	40	39	38
12f S	"	62	60	58	56	54	52	49	47	45	41	36	32	31	30	29	28
12e S	"	51	49	47	45	43	41	39	37	35	32	27	23	22	21	20	19
11g S	4 x 8	65	62	60	58	56	54	51	49	47	43	37	32	31	30	29	28
11f S	"	55	52	50	48	46	44	41	39	37	33	27	23	22	21	20	19
11e S	"	45	43	41	39	37	35	33	31	29	26	21	17	16	15	14	13
11d S	"	34	32	30	28	26	24	22	20	18	15	12	9	8	7	6	5
10f S	3 1/2 x 7	46	44	42	40	38	37	35	33	31	28	23	19	18	17	16	15
10e S	"	38	36	34	32	30	29	27	25	23	20	16	13	12	11	10	9
10d S	"	29	27	26	24	23	22	20	19	17	14	11	8	7	6	5	4

Bolts 3/4-in. diam. at 6-in. pitch.

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160.

Safe loads are in accordance with the working stresses prescribed by the London County Council (General Powers) Act, 1900, for stanchions of mild steel having "both ends fixed."

For other conditions and formulae, see notes commencing page 192 L.

Safe loads printed in italics are for heights greater than 40D.

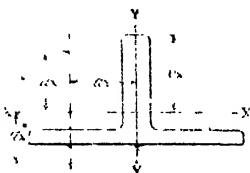
For explanations of properties, &c., see Part IV.

## REDPATH, BROWN &amp; CO., LIMITED.

COMPOUND STANCHIONS  
(or STRUTS).

Two Steel Equal Angles Back to Back.

Dimensions and Properties.



Composed of Two Equal Angles	Weight per foot in lbs.	Area in square inches.	Distance C <sub>x</sub> inches.	Radii of Gyration.		Eccentricity Coefficients	
				Axis Y-Y	Axis X-X	Axis Y-Y	Axis X-X
6 x 6 x $\frac{3}{8}$	59	16.88	4.24	2.53	1.81	1 + 0.94a <sub>v</sub>	1 + 1.29a <sub>x</sub>
" x $\frac{3}{8}$	49 $\frac{1}{2}$	14.22	4.29	2.50	1.83	1 + 0.96a <sub>v</sub>	1 + 1.28a <sub>x</sub>
" x $\frac{1}{2}$	40 $\frac{1}{2}$	11.50	4.34	2.48	1.84	1 + 0.98a <sub>v</sub>	1 + 1.28a <sub>x</sub>
5 x 5 x $\frac{3}{8}$	48 $\frac{1}{2}$	13.87	3.49	2.12	1.49	1 + 1.11a <sub>v</sub>	1 + 1.56a <sub>x</sub>
" x $\frac{3}{8}$	41	11.72	3.54	2.10	1.51	1 + 1.13a <sub>v</sub>	1 + 1.55a <sub>x</sub>
" x $\frac{1}{2}$	33 $\frac{1}{2}$	9.50	3.58	2.08	1.52	1 + 1.16a <sub>v</sub>	1 + 1.55a <sub>x</sub>
4 $\frac{1}{2}$ x 4 $\frac{1}{2}$ x $\frac{3}{8}$	43	12.38	3.11	1.92	1.34	1 + 1.21a <sub>v</sub>	1 + 1.74a <sub>x</sub>
" x $\frac{3}{8}$	36 $\frac{1}{2}$	10.47	3.16	1.90	1.35	1 + 1.24a <sub>v</sub>	1 + 1.73a <sub>x</sub>
" x $\frac{1}{2}$	29 $\frac{1}{2}$	8.50	3.21	1.88	1.36	1 + 1.28a <sub>v</sub>	1 + 1.72a <sub>x</sub>
4 x 4 x $\frac{3}{8}$	38	10.87	2.74	1.73	1.18	1 + 1.34a <sub>v</sub>	1 + 1.97a <sub>x</sub>
" x $\frac{3}{8}$	32	9.22	2.78	1.70	1.19	1 + 1.38a <sub>v</sub>	1 + 1.96a <sub>x</sub>
" x $\frac{1}{2}$	26 $\frac{1}{2}$	7.60	2.83	1.68	1.20	1 + 1.42a <sub>v</sub>	1 + 1.95a <sub>x</sub>
" x $\frac{3}{4}$	20	5.72	2.88	1.66	1.22	1 + 1.46a <sub>v</sub>	1 + 1.93a <sub>x</sub>
3 $\frac{1}{2}$ x 3 $\frac{1}{2}$ x $\frac{3}{8}$	28	7.97	2.41	1.50	1.03	1 + 1.55a <sub>v</sub>	1 + 2.25a <sub>x</sub>
" x $\frac{3}{8}$	23	6.50	2.45	1.48	1.05	1 + 1.60a <sub>v</sub>	1 + 2.23a <sub>x</sub>
" x $\frac{1}{2}$	17 $\frac{1}{2}$	4.97	2.50	1.45	1.06	1 + 1.65a <sub>v</sub>	1 + 2.22a <sub>x</sub>

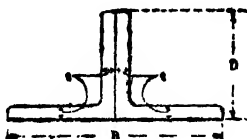
In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2 $\frac{1}{2}$  per cent. over this must be allowed. See page 7.

Each weight per foot is for the riveted shaft only. Weight of connections, &c., to be added.  
Least radii of gyration and relative eccentricity coefficients are printed in prominent type.  
We = actual eccentric load; K = relative eccentricity coefficient; W<sub>e</sub> = equivalent concentric value; W<sub>c</sub> = W x K.

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for a<sub>v</sub> and a<sub>x</sub> respectively.

For full explanations of tables, see notes commencing page 192 L.

REDPATH, BROWN & CO., LIMITED.



**COMPOUND STANCHIONS  
(or STRUTS).**

**Two Steel Equal Angles Back to Back.**

Safe Concentric Loads, in Tons.  
Ends Fixed.

Reference Mark.	Size, D x B inches.	HEIGHTS IN FEET.									
		2	3	4	5	6	7	8	9	10	12
9/S	3 x 6	38.9	36.6	34.3	32.0	29.6	27.3	25.0	22.7	20.4	
9c S	"	32.0	30.1	28.3	26.4	24.6	22.7	20.9	19.0	17.2	
9d S	"	24.5	23.1	21.7	20.3	18.9	17.5	16.1	14.7	13.3	10.5
9e S	"	20.4	19.2	18.1	16.9	15.8	14.6	13.5	12.3	11.2	8.9
9f S	"	16.9	15.9	15.0	14.0	13.1	12.1	11.2	10.2	9.2	7.3
7e S	2½ x 5	25.5	23.7	21.8	20.0	18.1	16.3	14.4	12.6		
7d S	"	19.3	17.9	16.5	15.2	13.8	12.4	11.0	9.6		
7c S	"	16.5	15.3	14.2	13.0	11.8	10.7	9.5	8.4	7.2	
7b S	"	13.1	12.1	11.2	10.3	9.4	8.5	7.5	6.6	5.7	
6c S	2½ x 4½	12.3	11.3	10.3	9.3	8.3	7.4	6.4			
6b S	"	11.8	10.8	9.9	9.0	8.0	7.1	6.2	5.3		
5b S	2 x 4	10.4	9.4	8.4	7.5	6.5	5.6				
5a S	"	7.7	7.0	6.3	5.6	4.9	4.2	3.5			

Rivets ¾-in. diam. at 6-in. pitch.

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160. Safe loads are, in accordance with the working stresses prescribed by the London County Council (General Powers) Act, 1900, for stanchions of mild steel having "both ends fixed."

For other conditions and formulae, see notes commencing page 162 L.

Safe loads printed in italics are for heights greater than 40D.

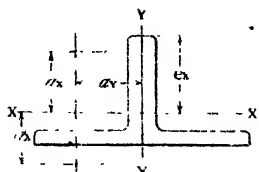
For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND STANCHIONS (or STRUTS).

Two Steel Equal Angles Back to Back.

Dimensions and Properties.



Composed of Two Equal Angles	Weight per foot in lbs.	Area in sq. in.	Distance between angles inches	Radii of Gyration		Eccentricity Coefficients	
				Axis Y-Y	Axis X-X	Axis Y-Y	Axis X-X
3 x 3 x $\frac{5}{16}$	23½	6.72	2.03	1.30	0.87	1 + 1.76a <sub>y</sub>	1 + 2.64a <sub>x</sub>
" x $\frac{3}{4}$	19½	5.50	2.08	1.28	0.89	1 + 1.82a <sub>y</sub>	1 + 2.62a <sub>x</sub>
" x $\frac{1}{2}$	15	4.22	2.12	1.26	0.90	1 + 1.89a <sub>y</sub>	1 + 2.60a <sub>x</sub>
" x $\frac{1}{4}$	13	3.55	2.15	1.24	0.91	1 + 1.93a <sub>y</sub>	1 + 2.60a <sub>x</sub>
" x $\frac{1}{8}$	10½	2.85	2.17	1.23	0.91	1 + 1.97a <sub>y</sub>	1 + 2.60a <sub>x</sub>
2½ x 2½ x $\frac{1}{2}$	16	4.40	1.70	1.08	0.73	1 + 2.14a <sub>y</sub>	1 + 3.18a <sub>x</sub>
" x $\frac{3}{4}$	12½	3.47	1.75	1.06	0.74	1 + 2.24a <sub>y</sub>	1 + 3.16a <sub>x</sub>
" x $\frac{1}{2}$	10	2.97	1.77	1.04	0.75	1 + 2.29a <sub>y</sub>	1 + 3.15a <sub>x</sub>
" x $\frac{1}{4}$	8½	2.37	1.80	1.05	0.75	1 + 2.55a <sub>y</sub>	1 + 3.15a <sub>x</sub>
2½ x 2½ x $\frac{1}{4}$	9½	2.26	1.54	0.94	0.67	1 + 2.51a <sub>y</sub>	1 + 3.51a <sub>x</sub>
" x $\frac{1}{8}$	7½	2.12	1.61	0.95	0.68	1 + 2.58a <sub>y</sub>	1 + 3.49a <sub>x</sub>
2 x 2 x $\frac{1}{2}$	7	1.88	1.42	0.83	0.59	1 + 2.88a <sub>y</sub>	1 + 3.97a <sub>x</sub>
" x $\frac{1}{4}$	5½	1.44	1.45	0.81	0.60	1 + 3.00a <sub>y</sub>	1 + 4.01a <sub>x</sub>

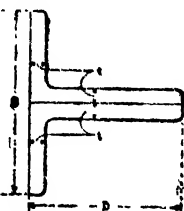
In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

Each weight per foot is for the riveted shaft only. Weight of connections, &c., to be added. Least radii of gyration and relative eccentricity coefficients are printed in prominent type. W<sub>e</sub> = actual eccentric load; K = relative eccentricity coefficient; W<sub>c</sub> = equivalent concentric value; W<sub>c</sub> = W<sub>e</sub> x K.

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for a<sub>y</sub> and a<sub>x</sub> respectively.

For full explanations of tables, see notes commencing page 192 L.

# REDPATH, BROWN & CO., LIMITED.



## COMPOUND STANCHIONS (or STRUTS).

Two Steel Unequal Angles Back to Back.

Short Legs Outstanding.

Safe Concentric Loads, in Tons.  
Ends Fixed.

Reference Mark.	Size, D x B Inches	HEIGHTS IN FEET.												
		2	3	4	5	6	7	8	9	10	12	14	16	
25g T	7 x 7	87.484	480.977	473.970	3.66	853.359	852.745	738.7						
25f T	"	74.171	1.68	0.65	0.62	0.58	0.55	0.52	0.49	8.43	8.37	7.31	7	
25e T	"	59.967	4.51	9.52	4.40	9.47	4.44	9.42	4.39	9.34	9.29	9		
21f T	6 x 8	71.569	2.66	9.64	5.62	2.59	9.57	6.55	3.52	9.48	3.43	7.39	0	
21e T	"	57.956	0.51	1.52	2.50	3.48	4.46	4.44	5.42	6.38	8.35	0.31	2	
20f T	6 x 7	66.964	2.61	7.59	1.56	5.53	9.51	3.48	7.46	1.41	0.36	9.30	6	
20e T	"	54.252	1.49	9.17	8.45	7.43	5.41	1.39	6.37	1.32	8.28	6.24	3	
20d T	"	41.239	5.37	8.36	2.34	5.32	9.31	2.29	6.27	9.24	6.21	3.18	0	
63f T	6 x 6	62.159	2.56	2.53	3.51	3.47	3.44	4.41	4.38	5.32	6.26	6		
63e T	"	50.347	9.45	4.42	9.40	5.36	0.35	6.33	1.30	7.25	7			
63d T	"	38.236	3.34	4.32	5.30	6.28	7.26	8.24	8.22	9.19	1			
Bore 1-in. diam. at 6-in. pitch.														

Rivets 1/4 in. diam. at 6-in. pitch.

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160.

Safe loads are in accordance with the working stresses prescribed by the London County Council (General Powers) Act, 1909, for stanchions of mild steel having "both ends fixed."

For other conditions and formulae, see notes commencing page 192 L.

For explanations of properties, &c., see Part IV.

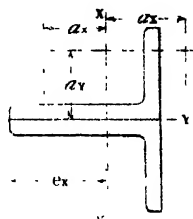
# REDPATH, BROWN & CO., LIMITED.

## COMPOUND STANCHIONS (or STRUTS).

Two Steel Unequal Angles Back to Back.

Short Legs Outstanding.

Dimensions and Properties.



Composed of Two Unequal Angles.	Weight per foot in lbs.	Area in square inches.	Distance $e_x$ inches.	Radius of gyration.		Eccentricity Coefficients.	
				Axis Y-Y	Axis X-X	Axis Y-Y	Axis X-X
$7 \times 3\frac{1}{2} \times \frac{3}{8}$	51	14.62	4.40	1.24	2.21	$1+2.26a_y$	$1+0.90a_x$
" $\times \frac{5}{8}$	43 $\frac{1}{2}$	12.34	4.45	1.22	2.22	$1+2.35a_y$	$1+0.90a_x$
" $\times \frac{7}{8}$	35 $\frac{1}{2}$	10.00	4.50	1.20	2.24	$1+2.44a_y$	$1+0.90a_x$
$6 \times 4 \times \frac{3}{8}$	41	11.72	3.98	1.51	1.88	$1+1.74a_y$	$1+1.12a_x$
" $\times \frac{5}{8}$	36	9.56	4.03	1.49	1.90	$1+1.80a_y$	$1+1.12a_x$
$6 \times 3\frac{1}{2} \times \frac{3}{8}$	39	11.16	3.89	1.28	1.89	$1+2.12a_y$	$1+1.09a_x$
" $\times \frac{5}{8}$	32	9.00	3.94	1.26	1.91	$1+2.20a_y$	$1+1.08a_x$
" $\times \frac{7}{8}$	24 $\frac{1}{2}$	6.80	3.99	1.24	1.91	$1+2.28a_y$	$1+1.09a_x$
$6 \times 3 \times \frac{3}{8}$	37	10.47	3.78	1.06	1.89	$1+2.66a_y$	$1+1.06a_x$
" $\times \frac{5}{8}$	30 $\frac{1}{2}$	8.50	3.83	1.04	1.91	$1+2.78a_y$	$1+1.05a_x$
" $\times \frac{7}{8}$	23 $\frac{1}{2}$	6.47	3.88	1.01	1.92	$1+2.91a_y$	$1+1.05a_x$

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2 $\frac{1}{2}$  per cent. over this must be allowed. See page 7.

Each weight per foot is for the riveted shaft only. Weight of connections, &c., to be added.

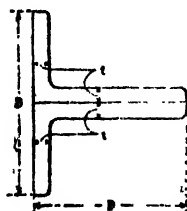
Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

W = actual eccentric load; K = relative eccentricity coefficient; Wc = equivalent concentric value;  $W_e = W \times K$ .

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for  $a_y$  and  $a_x$  respectively.

For full explanations of tables, see notes commencing page 102 L.

REDPATH, BROWN & CO., LIMITED.



**COMPOUND STANCHIONS  
(or STRUTS).**

**Two Steel Unequal Angles Back to Back.**

Short Legs Outstanding.

Safe Concentric Loads, in Tons.  
Ends Fixed.

Reference Mark.	Size, D × B inches.	HEIGHTS IN FEET.													
		2	3	4	5	6	7	8	9	10	12	14	16		
17f T	5 × 8	63.6	61.9	59.9	57.8	55.8	53.8	51.7	49.7	47.6	43.6	39.5	35.4		
17e T	"	51.0	50.3	48.7	47.0	45.4	43.7	42.1	40.5	38.8	35.5	32.0			
17d T	"	39.6	38.3	37.1	35.6	34.6	33.3	32.1	30.8	29.6	27.1	24.6	22.1		
15f T	5 × 6	55.0	52.5	50.1	47.6	45.2	42.7	40.3	37.8	35.4	30.5	25.5			
15e T	"	44.6	42.6	40.6	38.8	36.5	34.1	31.8	29.4	27.3	22.0	17.2			
15d T	"	33.9	32.4	31.0	29.7	28.6	27.4	26.2	25.0	23.8	18.1	14.9			
11e T	4 × 6	38.9	37.3	35.5	33.4	31.2	29.0	26.7	24.5	22.4	17.9				
11d T	"	29.7	28.4	27.1	25.8	24.6	23.3	22.0	20.7	19.4	16.8	14.3			
7d T	3 × 5	22.4	21.2	19.9	18.6	17.4	16.1	14.9	13.6	12.4	9.9				
7e T	"	18.9	17.9	16.8	15.8	14.7	13.7	12.6	11.6	10.5	8.4				
Rivets 3-in. diam. at 6-in. pitch.															

Rivets  $\frac{3}{4}$ -in. diam. at 6-in. pitch.

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160.

Safe loads are in accordance with the working stresses prescribed by the London County Council (General Powers) Act, 1909, for stanchions of mild steel having "both ends fixed."

For other conditions and formulae, see notes commencing page 192 L.

Safe loads printed in italics are for heights greater than 10D.

For explanations of properties, &c., see Part IV.

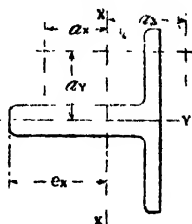
REDPATH, BROWN & CO., LIMITED.

**COMPOUND STANCHIONS  
(or STRUTS).**

**Two Steel Unequal Angles Back to Back.**

**Short Legs Outstanding.**

**Dimensions and Properties.**



Composed of Two Unequal Angles.	Weight per foot in lbs	Area in square inches.	Distance $e_x$ inches.	Radii of Gyration.		Eccentricity Coefficients.	
				Axis Y—Y	Axis X—X	Axis Y—Y	Axis X—X
$5 \times 4 \times \frac{3}{8}$	37	10.47	3.39	1.60	1.54	$1 + 1.56a_y$	$1 + 1.43a_x$
" $\times \frac{1}{2}$	30½	8.50	3.44	1.58	1.55	$1 + 1.61a_y$	$1 + 1.43a_x$
" $\times \frac{3}{4}$	23½	6.47	3.49	1.55	1.57	$1 + 1.65a_y$	$1 + 1.42a_x$
$5 \times 3 \times \frac{3}{8}$	32½	9.22	3.22	1.13	1.56	$1 + 2.36a_y$	$1 + 1.32a_x$
" $\times \frac{1}{2}$	27	7.50	3.26	1.10	1.58	$1 + 2.46a_y$	$1 + 1.31a_x$
" $\times \frac{3}{4}$	21	5.72	3.32	1.08	1.59	$1 + 2.57a_y$	$1 + 1.31a_x$
$4 \times 3 \times \frac{1}{2}$	23	6.50	2.68	1.18	1.24	$1 + 2.14a_y$	$1 + 1.75a_x$
" $\times \frac{3}{4}$	17½	4.97	2.73	1.16	1.25	$1 + 2.23a_y$	$1 + 1.75a_x$
$3 \times 2\frac{1}{2} \times \frac{3}{8}$	14	3.84	2.05	1.00	0.92	$1 + 2.47a_y$	$1 + 2.43a_x$
" $\times \frac{1}{2}$	12	3.24	2.08	0.99	0.92	$1 + 2.53a_y$	$1 + 2.37a_x$

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

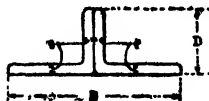
Each weight per foot is for the riveted shaft only. Weight of connections, &c., to be added.  
Least radii of gyration and relative eccentricity coefficients are printed in prominent type.  
 $W_e$  = actual eccentric load;  $K$  = relative eccentricity coefficient;  $W_c$  = equivalent concentric value;  $W_c = W_e \times K$ .

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for  $a_x$  and  $a_y$  respectively.

For full explanations of tables, see notes commencing page 192 L.

REDPATH, BROWN & CO., LIMITED.

**COMPOUND STANCHIONS  
(or STRUTS).**



**Two Steel Unequal Angles Back to Back.**

**Long Legs Outstanding.**

**Safe Concentric Loads, in Tons.  
Ends Fixed.**

Reference Mark.	Size, B x B inches.	HEIGHTS IN FEET.													
		2	3	4	5	6	7	8	9	10	11	12	14		
25g U	3½ x 14	85	350	475	570	765	860	956	151	248	441	536	6		
25f U	"	72	163	063	959	955	851	847	743	739	635	531	5		
25e U	"	58	555	252	048	745	542	339	035	832	529	226	0		
21f U	4 x 12	69	966	763	660	557	354	251	047	944	841	638	532	2	
21e U	"	56	754	251	749	146	644	141	639	136	634	031	526	6	
20f U	3½ x 12	65	161	558	054	551	047	543	940	436	933	429	9		
20e U	"	52	950	047	244	441	638	835	933	130	327	524	7		
20d U	"	40	238	136	033	931	729	627	525	423	321	119	0		
63f U	3 x 12	59	955	851	847	743	639	635	531	427	4				
63e U	"	48	745	542	239	035	732	529	226	022	7				
63d U	"	37	234	732	329	927	425	022	520	117	6				

Nuts 1-in. diam. at 6-in. pitch.

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 180.

Safe loads are in accordance with the working stresses prescribed by the London County Council (General Powers) Act, 1900, for stanchions of mild steel having "both ends fixed."

For other conditions and formulae, see notes commencing page 192 L.

Safe loads printed in italics are for heights greater than 40D.

For explanations of properties, &c., see Part IV.

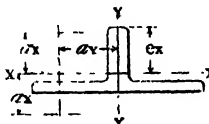
# REDPATH, BROWN & CO., LIMITED.

## COMPOUND STANCHIONS (or STRUTS).

Two Steel Unequal Angles Back to Back.

Long Legs Outstanding.

Dimensions and Properties.



Composed of Two Unequal Angles.	Weight per foot in lbs.	Area in square inches.	Distance $e_x$ inches.	Radii of Gyration.		Eccentricity Coefficients.	
				Axis Y-Y	Axis X-X	Axis Y-Y	Axis X-X
$7 \times 3\frac{1}{2} \times \frac{1}{2}$	50 $\frac{1}{2}$	14.62	2.64	3.41	0.90	$1+0.60a_y$	$1+8.24a_x$
" $\times \frac{3}{8}$	43	12.34	2.69	3.38	0.91	$1+0.61a_y$	$1+8.22a_x$
" $\times \frac{1}{2}$	35	10.00	2.74	3.35	0.92	$1+0.62a_y$	$1+8.20a_x$
$6 \times 4 \times \frac{3}{8}$	40 $\frac{1}{2}$	11.72	2.98	2.76	1.12	$1+0.79a_y$	$1+2.37a_x$
" $\times \frac{1}{2}$	33	9.50	3.03	2.73	1.13	$1+0.81a_y$	$1+2.36a_x$
$6 \times 3\frac{1}{2} \times \frac{3}{8}$	38 $\frac{1}{2}$	11.10	2.63	2.83	0.94	$1+0.75a_y$	$1+2.93a_x$
" $\times \frac{1}{2}$	31 $\frac{1}{2}$	9.00	2.68	2.81	0.96	$1+0.76a_y$	$1+2.91a_x$
" $\times \frac{3}{8}$	24	6.85	2.73	2.77	0.97	$1+0.78a_y$	$1+2.90a_x$
$6 \times 3 \times \frac{3}{8}$	36 $\frac{1}{2}$	10.47	2.27	2.92	0.77	$1+0.71a_y$	$1+3.80a_x$
" $\times \frac{1}{2}$	29 $\frac{1}{2}$	8.50	2.32	2.89	0.78	$1+0.72a_y$	$1+3.76a_x$
" $\times \frac{3}{8}$	23	6.47	2.37	2.86	0.79	$1+0.73a_y$	$1+3.74a_x$

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2 $\frac{1}{2}$  per cent. over this must be allowed. See page 7.

Each weight per foot is for the riveted shaft only. Weight of connections, &c., to be added.

Least radii of gyration and relative eccentricity coefficients are printed in prominent type.  $W_e$ =actual eccentric load;  $K$ =relative eccentricity coefficient;  $W_o$ =equivalent concentric value;  $W_o = W_e \times K$ .

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for  $a_y$  and  $a_x$  respectively.

For full explanations of tables, see notes commencing page 192L.

REDPATH, BROWN & CO., LIMITED.

COMPOUND STANCHIONS  
(or STRUTS).



Two Steel Unequal Angles Back to Back.

Long Legs Outstanding.

Safe Concentric Loads, in Tons.  
Ends Fixed.

Reference Mark.	Size, D x B inches.	HEIGHTS IN FEET.													
		2	3	4	5	6	7	8	9	10	11	12	14		
17f U	4 x 10	82.6	59.9	57.2	54.4	51.7	49.0	46.3	43.6	40.9	38.1	35.4	30.0		
17e U	"	50.9	48.7	46.5	44.3	42.2	40.0	37.8	35.6	33.4	31.3	29.1	24.7		
17d U	"	53.8	51.7	49.5	47.3	45.1	42.9	40.7	38.5	36.3	34.1	31.9	27.6		
15f U	3 x 10	53.0	49.6	46.2	42.7	39.3	35.9	32.5	29.0	25.6					
15e U	"	43.2	40.5	37.7	34.9	32.2	29.5	26.7	23.9	21.2					
15d U	"	33.1	31.1	28.9	26.8	24.7	22.7	20.6	18.5	16.5	14.4				
11e U	3 x 8	37.6	35.4	33.1	30.8	28.5	26.2	23.9	21.7	19.4	17.1				
11d U	"	28.8	27.1	25.4	23.7	21.9	20.2	18.5	16.8	15.1	13.4				
7d U	2½ x 6	21.7	20.2	18.6	17.0	15.4	13.8	12.2	10.7						
7c U	"	18.4	17.0	15.7	14.4	13.1	11.7	10.4	9.1						

Rivets ½-in. diam. at 6-in. pitch.

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 180.

Safe loads are in accordance with the working stresses prescribed by the London County Council (General Powers) Act, 1900, for stanchions of mild steel having "both ends fixed."

For other conditions and formulæ, see notes commencing page 192 L.

Safe loads printed in italics are for heights greater than 40D.

For explanations of properties, &c., see Part IV.

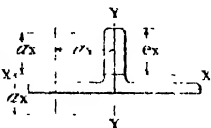
# REDPATH, BROWN & CO., LIMITED.

## COMPOUND STANCHIONS (or STRUTS).

Two Steel Unequal Angles Back to Back.

Long Legs Outstanding.

Dimensions and Properties.



Composed of Two Unequal Angles.	Weight per foot in lbs.	Area in square inches.	Distance $c_x$ inches.	Radii of Gyration.		Eccentricity Coefficients.	
				Axis Y-Y	Axis X-X	Axis Y-Y	Axis X-X
$5 \times 4 \times \frac{3}{8}$	36½	10.47	2.89	2.22	1.15	$1+1.01a_y$	$1+2.16a_x$
" $\times \frac{1}{2}$	29½	8.50	2.94	2.20	1.17	$1+1.03a_y$	$1+2.15a_x$
" $\times \frac{3}{4}$	23	6.47	2.99	2.18	1.18	$1+1.06a_y$	$1+2.14a_x$
$5 \times 3 \times \frac{3}{8}$	32	9.22	2.21	2.37	0.80	$1+0.89a_y$	$1+3.40a_x$
" $\times \frac{1}{2}$	26½	7.50	2.26	2.34	0.82	$1+0.91a_y$	$1+2.99a_x$
" $\times \frac{3}{4}$	20	5.72	2.31	2.31	0.83	$1+0.93a_y$	$1+2.97a_x$
$4 \times 3 \times \frac{1}{2}$	23	6.50	2.18	1.80	0.85	$1+1.23a_y$	$1+3.41a_x$
" $\times \frac{3}{4}$	17½	4.97	2.23	1.78	0.86	$1+1.26a_y$	$1+3.40a_x$
$3 \times 2\frac{1}{2} \times \frac{3}{8}$	14	3.84	1.80	1.32	0.72	$1+1.73a_y$	$1+3.37a_x$
" $\times \frac{1}{2}$	12	3.24	1.83	1.30	0.73	$1+1.76a_y$	$1+3.35a_x$

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

Each weight per foot is for the riveted shaft only. Weight of connections, &c., to be added.

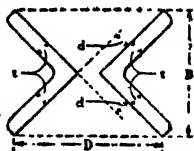
Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

$W_e$  = actual eccentric load;  $K$  = relative eccentricity coefficients;  $W_c$  = equivalent concentric value;  $W_c = W_e \times K$ .

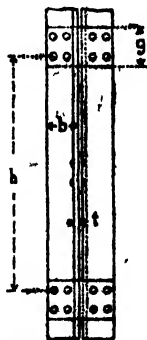
In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for  $a_y$  and  $a_x$  respectively.

For full explanations of tables, see notes commencing page 192 L.

## REDPATH, BROWN &amp; CO., LIMITED.

**COMPOUND STANCHIONS  
(or STRUTS).****Two Steel Equal Angles Battened.****Safe Concentric Loads, in Tons.  
Ends Fixed.**

Reference Mark.	Size, D x B inches.	HEIGHTS IN FEET.													
		2	3	4	5	6	7	8	9	10	12	14	16		
14g V	10 $\frac{3}{8}$ x 8 $\frac{1}{2}$	105	103	100	98	96	94	291	989	787	583	178	674	2	
14f V	10 x "	88	786	985	083	181	379	477	675	773	970	286	582	7	
14e V	9 $\frac{1}{8}$ x "	71	870	368	867	365	864	362	961	459	956	953	951	0	
13g V	9 x 7 $\frac{1}{8}$	85	783	581	379	176	974	772	570	368	163	659	254	8	
13f V	8 $\frac{3}{4}$ x "	72	470	668	766	965	063	261	359	557	653	950	246	5	
13e V	8 $\frac{1}{4}$ x "	58	757	355	854	352	851	349	848	346	043	940	937	9	
12g V	8 $\frac{1}{8}$ x 6 $\frac{3}{8}$	76	073	871	669	467	265	062	860	658	454	049	645	2	
12f V	7 $\frac{1}{2}$ x "	64	362	560	658	856	955	153	351	449	645	942	238	5	
12e V	7 $\frac{1}{8}$ x "	52	350	849	347	846	344	943	441	940	437	434	531	5	

For 6-in. and 5-in.  
angles.For 4 1/2-in. to 2-in.  
angles.

The angles forming stanchions or struts of this class are usually secured together with batten plates spaced alternately at right angles to each other.

See opposite page for conventional spacing and proportions.

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160.

Safe loads are, in accordance with the working stresses prescribed by the London County Council General Powers Act, 1900, for stanchions of mild steel having "both ends fixed."

For other conditions and formulae, see notes commencing page 191.

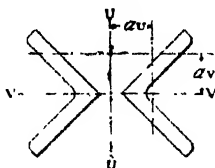
For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND STANCHIONS (or STRUTS).

### Two Steel Equal Angles Battened.

#### Dimensions and Properties.



Composed of Two Equal Angles.	Weight per foot in lbs.	Area in square inches.	Thickness of Batten Plates. Inch.	Radii of Gyration.		Eccentricity Coefficients.	
				Axis V-V	Axis U-U	Axis V-V	Axis U-U
6 x 6 x $\frac{3}{8}$	57 $\frac{1}{2}$	16.88	$\frac{3}{8}$	2.28	3.23	1 + 0.82av	1 + 0.50au
" x $\frac{3}{8}$	48 $\frac{1}{2}$	14.22	$\frac{3}{8}$	2.30	3.09	1 + 0.81av	1 + 0.52au
" x $\frac{3}{8}$	39 $\frac{1}{2}$	11.50	$\frac{3}{8}$	2.32	2.95	1 + 0.79av	1 + 0.55au
5 x 5 x $\frac{3}{8}$	47 $\frac{1}{2}$	13.87	$\frac{3}{8}$	1.88	2.83	1 + 1.00av	1 + 0.56au
" x $\frac{3}{8}$	40	11.72	$\frac{3}{8}$	1.89	2.69	1 + 0.98av	1 + 0.59au
" x $\frac{3}{8}$	32 $\frac{1}{2}$	9.50	$\frac{3}{8}$	1.92	2.55	1 + 0.96av	1 + 0.63au
4 $\frac{1}{2}$ x 4 $\frac{1}{2}$ x $\frac{3}{8}$	42 $\frac{1}{2}$	12.38	$\frac{3}{8}$	1.69	2.63	1 + 1.12av	1 + 0.60au
" x $\frac{3}{8}$	36	10.47	$\frac{3}{8}$	1.70	2.49	1 + 1.10av	1 + 0.64au
" x $\frac{3}{8}$	29	8.50	$\frac{3}{8}$	1.72	2.35	1 + 1.07av	1 + 0.69au

#### CONVENTIONAL MAXIMUM SPACING AND MINIMUM PROPORTIONS OF BATTEN PLATES FOR CONCENTRIC LOADING (*Am. Ry. Engineering and Maintenance of Way Assoc.*)

Maximum centres of end rivets of batten plates =  $\lambda$  inches.

$\lambda$  = the lesser value of  $\left\{ \begin{array}{l} 10 \text{ times } b \text{ the width of one leg in inches.} \\ 60 \text{ times } t \text{ the angle thickness in inches.} \end{array} \right.$

Minimum width of batten plates =  $g$  inches.

$g$  = the greater value of  $\left\{ \begin{array}{l} b \text{ the width of one leg, or } c \text{ the horizontal centres of rivets, or the} \\ \text{least width suitable for 2 rivets, in inches.} \end{array} \right.$

Rivet diameter =  $\frac{3}{4}$  inch for angles  $\frac{3}{8}$ ,  $\frac{1}{2}$ , and  $\frac{3}{4}$  inch thick.

" " =  $\frac{1}{2}$  " " "  $\frac{1}{4}$  inch thick.

" " =  $\frac{1}{4}$  " " "  $\frac{1}{8}$  and  $\frac{1}{16}$  inch thick.

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of  $2\frac{1}{2}$  per cent. ov. this must be allowed. See page 7.

Each weight per foot is for the shaft only. Weight of batten plates, rivets, base, &c., to be added.

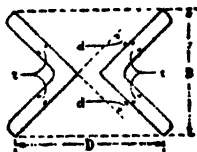
Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

Unsymmetrical eccentric load:  $K$  = relative eccentricity coefficient;  $W$  = equivalent concentric value, Wm & Wm K.

In axial eccentricity coefficient substitute actual value of "arm of eccentricity" for  $av$  and  $au$  respectively.

For full explanations of tables, see notes commencing page 182 L.

## REDPATH, BROWN &amp; CO., LIMITED.

COMPOUND STANCHIONS  
(or STRUTS).

Two Steel Equal Angles Battened.

Safe Concentric Loads, in Tons.  
Ends Fixed.

Reference Mark.	Size, D x B inches.	HEIGHTS IN FEET.													
		2	3	4	5	6	7	8	9	10	11	12	14		
11g V	$7\frac{1}{2} \times 5\frac{1}{2}$	66.2	64.0	61.9	59.7	57.5	55.3	53.1	50.9	48.7	46.5	44.3	39.9		
11f V	$7\frac{1}{2} \times "$	56.2	54.3	52.5	50.7	48.8	47.0	45.1	43.3	41.5	39.6	37.8	34.1		
11e V	$6\frac{1}{2} \times "$	45.7	44.3	42.8	41.3	39.8	38.4	36.9	35.4	33.9	32.4	31.0	28.0		
11d V	$6\frac{1}{2} \times "$	34.9	33.8	32.7	31.5	30.4	29.3	28.2	27.1	26.0	24.9	23.8	21.5		
10f V	$6\frac{1}{2} \times 5$	48.1	46.2	44.4	42.5	40.7	38.9	37.0	35.2	33.3	31.5	29.6	25.9		
10e V	$6\frac{1}{2} \times "$	39.3	37.8	36.3	34.8	33.3	31.8	30.3	28.8	27.2	25.7	24.2	21.5		
10d V	$5\frac{1}{2} \times "$	30.0	28.9	27.8	26.7	25.6	24.5	23.4	22.2	21.1	20.0	18.9	16.7		
9f V	$5\frac{1}{2} \times 4\frac{1}{2}$	40.0	38.1	36.3	34.5	32.6	30.8	28.9	27.1	25.3	23.4	21.6	17.9		
9e V	$5\frac{1}{2} \times "$	32.8	31.3	29.8	28.4	26.9	25.4	23.9	22.4	21.0	19.5	18.0	15.1		
9d V	$5\frac{1}{2} \times "$	25.2	24.1	22.9	21.8	20.7	19.6	18.5	17.4	16.3	15.1	14.0	11.8		
9c V	$5\frac{1}{2} \times "$	21.2	20.3	19.3	18.4	17.5	16.5	15.6	14.7	13.8	12.8	11.9	10.0		
9b V	$4\frac{1}{2} \times "$	17.2	16.4	15.7	14.9	14.2	13.4	12.7	11.9	11.2	10.4	9.7	8.2		
7e V	$4\frac{1}{2} \times 3\frac{1}{2}$	26.2	24.8	23.3	21.8	20.3	18.9	17.4	15.9	14.4	13.0	11.5			
7d V	$4\frac{1}{2} \times "$	20.3	19.2	18.0	16.9	15.8	14.7	13.6	12.5	11.4	10.3	9.2			
7c V	$4\frac{1}{2} \times "$	17.1	16.2	15.2	14.3	13.4	12.5	11.5	10.6	9.7	8.8	7.8			
7b V	$4\frac{1}{2} \times "$	13.9	13.1	12.4	11.7	10.9	10.2	9.4	8.7	7.9	7.2	6.4			
6c V	$4\frac{1}{2} \times 3\frac{1}{2}$	15.1	14.2	13.3	12.3	11.4	10.5	9.6	8.6	7.7	6.8				
6b V	$3\frac{1}{2} \times "$	12.3	11.5	10.8	10.0	9.3	8.5	7.8	7.0	6.3	5.5				
5b V	$3\frac{1}{2} \times 2\frac{1}{2}$	10.7	9.9	9.1	8.4	7.6	6.9	6.1	5.3						
5a V	$3\frac{1}{2} \times "$	8.2	7.6	7.0	6.4	5.9	5.3	4.7	4.2	3.6					

For sketch, see page 180 L.

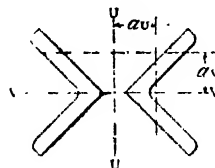
The above safe loads are calculated for ratios of slenderness up to, but not exceeding 160.  
 Safe loads are in accordance with the working stresses prescribed by the London County Council (General Powers Act, 1866, for stanchions of mild steel having "both ends fixed."  
 For other conditions and formulae, see notes commencing page 182 L.  
 Safe loads printed in Italics are for heights greater than 405.  
 For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## COMPOUND STANCHIONS (or STRUTS).

Two Steel Equal Angles Battened.

Dimensions and Properties.



Composed of Two Equal Angles.	Weight per foot in lbs.	Area in square inches.	Thickness of Batten Plate. Inch.	Radii of Gyration.		Eccentricity Coefficients.	
				Axis V-V	Axis U-U	Axis V-V	Axis U-U
4 x 4 x $\frac{3}{8}$	37	10.87	$\frac{3}{8}$	1.48	2.44	1+1.28a <sub>v</sub>	1+0.64a <sub>u</sub>
" x $\frac{3}{8}$	31½	9.22	$\frac{3}{8}$	1.50	2.29	1+1.28a <sub>v</sub>	1+0.69a <sub>u</sub>
" x $\frac{3}{8}$	25½	7.50	$\frac{3}{8}$	1.52	2.15	1+1.22a <sub>v</sub>	1+0.74a <sub>u</sub>
" x $\frac{3}{8}$	19½	5.72	$\frac{3}{8}$	1.54	2.01	1+1.19a <sub>v</sub>	1+0.81a <sub>u</sub>
3½ x 3½ x $\frac{5}{16}$	27½	7.97	$\frac{5}{16}$	1.29	2.10	1+1.47a <sub>v</sub>	1+0.74a <sub>u</sub>
" x $\frac{5}{16}$	22½	6.50	$\frac{5}{16}$	1.31	1.95	1+1.43a <sub>v</sub>	1+0.81a <sub>u</sub>
" x $\frac{5}{16}$	17	4.97	$\frac{5}{16}$	1.34	1.81	1+1.38a <sub>v</sub>	1+0.89a <sub>u</sub>
3 x 3 x $\frac{5}{16}$	23	6.72	$\frac{5}{16}$	1.09	1.90	1+1.76a <sub>v</sub>	1+0.81a <sub>u</sub>
" x $\frac{5}{16}$	19	5.50	$\frac{5}{16}$	1.12	1.76	1+1.70a <sub>v</sub>	1+0.89a <sub>u</sub>
" x $\frac{5}{16}$	14½	4.22	$\frac{5}{16}$	1.13	1.61	1+1.64a <sub>v</sub>	1+0.99a <sub>u</sub>
" x $\frac{5}{16}$	12½	3.55	$\frac{5}{16}$	1.15	1.58	1+1.61a <sub>v</sub>	1+1.01a <sub>u</sub>
" x $\frac{5}{16}$	10	2.88	$\frac{5}{16}$	1.15	1.47	1+1.60a <sub>v</sub>	1+1.11a <sub>u</sub>
2½ x 2½ x $\frac{1}{2}$	15½	4.50	$\frac{1}{2}$	0.91	1.56	1+2.12a <sub>v</sub>	1+0.99a <sub>u</sub>
" x $\frac{1}{2}$	12	3.47	$\frac{1}{2}$	0.93	1.41	1+2.02a <sub>v</sub>	1+1.11a <sub>u</sub>
" x $\frac{1}{2}$	10	2.92	$\frac{1}{2}$	0.94	1.38	1+1.97a <sub>v</sub>	1+1.14a <sub>u</sub>
" x $\frac{1}{2}$	8½	2.37	$\frac{1}{2}$	0.95	1.26	1+1.94a <sub>v</sub>	1+1.28a <sub>u</sub>
2½ x 2½ x $\frac{1}{4}$	9	2.26	$\frac{1}{4}$	0.84	1.28	1+2.22a <sub>v</sub>	1+1.22a <sub>u</sub>
" x $\frac{1}{4}$	7½	2.12	$\frac{1}{4}$	0.85	1.17	1+2.20a <sub>v</sub>	1+1.37a <sub>u</sub>
2 x 2 x $\frac{1}{2}$	6½	1.88	$\frac{1}{2}$	0.74	1.07	1+2.56a <sub>v</sub>	1+1.48a <sub>u</sub>
" x $\frac{1}{2}$	5	1.44	$\frac{1}{2}$	0.75	1.03	1+2.49a <sub>v</sub>	1+1.54a <sub>u</sub>

For conventional spacing and proportions, see page 181 L.

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

Each weight per foot is for the shaft only. Weight of batten plates, rivets, base, &c., to be added.

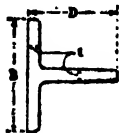
Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

W=actual eccentric load; K=relative eccentricity coefficient; W<sub>e</sub>=equivalent concentric value; W<sub>max</sub>=W x K.

In axial eccentricity coefficient substitute actual value of "arm of eccentricity" for a<sub>v</sub> and a<sub>u</sub> respectively.

For full explanations of tables see notes commencing page 182 L.

## REDPATH, BROWN &amp; CO., LIMITED.


**STANCHIONS (or STRUTS).**  
**Steel Tees.**

Safe Concentric Loads, in Tons.  
Ends Fixed.

Reference Mark.	Size, B × D × t inches.	HEIGHTS IN FEET.												
		2	3	4	5	6	7	8	9	10	11	12	14	
21e W	6 × 4 × $\frac{1}{2}$	28.4	27.2	25.9	24.6	23.4	22.1	20.8	19.5	18.3	17.0	15.7	13.9	
20e W	6 × 3 × $\frac{1}{2}$	21.5	22.8	21.2	19.6	17.9	16.3	14.7	13.0	11.4				
20d W	" × $\frac{3}{8}$	18.7	17.5	16.2	15.0	13.8	12.5	11.3	10.1	8.8				
19e W	5 × 4 × $\frac{1}{2}$	25.3	24.2	23.0	21.8	20.6	19.4	18.3	17.1	15.9	14.7	13.5	11.9	
19d W	" × $\frac{3}{8}$	19.3	18.4	17.5	16.5	15.6	14.7	13.8	12.9	11.9	11.0	10.1	8.5	
17e W	5 × 3 × $\frac{1}{2}$	21.6	20.3	18.9	17.5	16.1	14.7	13.4	12.0	10.6				
17d W	" × $\frac{3}{8}$	16.6	15.5	14.5	13.4	12.4	11.3	10.3	9.3	8.2	7.2			
16e W	4 × 5 × $\frac{1}{2}$	24.4	22.7	21.1	19.4	17.8	16.2	14.5	12.9	11.2				
16d W	" × $\frac{3}{8}$	18.5	17.2	16.0	14.7	13.4	12.1	10.8	9.6	8.3				

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 180. Safe loads are, in accordance with the working stresses prescribed by the London County Council (General Powers) Act, 1909, for stanchions of mild steel having "both ends fixed."

For other conditions and formulæ, see notes commencing page 192 L.

Safe loads printed in italics are for heights greater than 40D.

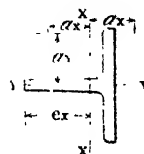
For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## STANCHIONS (or STRUTS).

### Steel Tees.

#### Dimensions and Properties.



Size, B x D x t inches.	Weight per foot in lbs.	Area in square inches.	Distance $e_x$ inches.	Radii of Gyration.		Eccentricity Coefficients.	
				Axis Y-Y	Axis X-X	Axis Y-Y	Axis X-X
6 x 4 x 1/2	16.22	4.771	3.03	1.34	1.13	1 + 1.66a <sub>y</sub>	1 + 2.39a <sub>x</sub>
6 x 3 x 1/2	14.53	4.272	2.32	1.42	0.78	1 + 1.48a <sub>y</sub>	1 + 3.76a <sub>x</sub>
" x 3/4	11.08	3.269	2.37	1.40	0.79	1 + 1.53a <sub>y</sub>	1 + 3.75a <sub>x</sub>
5 x 4 x 1/2	14.51	4.268	2.95	1.08	1.16	1 + 2.13a <sub>y</sub>	1 + 2.18a <sub>x</sub>
" x 3/4	11.07	3.257	3.00	1.06	1.17	1 + 2.21a <sub>y</sub>	1 + 2.19a <sub>x</sub>
5 x 3 x 1/2	12.79	3.762	2.26	1.15	0.82	1 + 1.87a <sub>y</sub>	1 + 3.37a <sub>x</sub>
" x 3/4	9.78	2.875	2.31	1.13	0.83	1 + 1.94a <sub>y</sub>	1 + 3.37a <sub>x</sub>
4 x 5 x 1/2	14.50	4.264	3.47	0.78	1.56	1 + 3.31a <sub>y</sub>	1 + 1.43a <sub>x</sub>
" x 3/4	11.06	3.253	3.53	0.76	1.54	1 + 3.45a <sub>y</sub>	1 + 1.48a <sub>x</sub>

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2 1/2 per cent. over this must be allowed. See page 7.

Each weight per foot is for the shaft only. Weight of connections, &c., to be added.

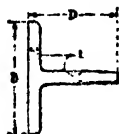
Least radii of gyration and relative eccentricity coefficients are printed in prominent type.

We=actual eccentric load; K=relative eccentricity coefficient; Wc=equivalent concentric value; Wc=We x K.

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for a<sub>y</sub> and a<sub>x</sub> respectively.

For full explanations of tables, see notes commencing page 192 L.

## REDPATH, BROWN &amp; CO., LIMITED.

**STANCHIONS (or STRUTS).****Steel Tees.**

Safe Concentric Loads, in Tons.  
Ends Fixed.

Reference Mark.	Size, B x D x t inches	HEIGHTS IN FEET.									
		2	3	4	5	6	7	8	9	10	11
15e W	4 x 4 x $\frac{1}{2}$	21.7	20.3	19.0	17.6	16.2	14.9	13.5	12.2	10.8	9.4
15d W	" x $\frac{3}{8}$	16.5	15.4	14.4	13.3	12.3	11.2	10.1	9.1	8.0	
14e W	4 x 3 x $\frac{1}{2}$	18.8	17.7	16.6	15.4	14.3	13.1	12.0	10.8	9.7	8.5
14d W	" x $\frac{3}{8}$	14.5	13.6	12.7	11.8	11.0	10.1	9.2	8.4	7.5	6.6
13e W	3 $\frac{1}{2}$ x 3 $\frac{1}{2}$ x $\frac{1}{2}$	18.5	17.1	15.8	14.5	13.1	11.8	10.5	9.1		
13d W	" x $\frac{3}{8}$	14.1	13.0	12.0	11.0	9.9	8.9	7.8	6.8		
11e W	3 x 3 x $\frac{1}{2}$	15.3	14.0	12.7	11.4	10.1	8.8	7.5			
11d W	" x $\frac{3}{8}$	11.7	10.7	9.6	8.6	7.6	6.6	5.5			
8d W	2 $\frac{1}{2}$ x 2 $\frac{1}{2}$ x $\frac{3}{8}$	9.3	8.3	7.3	6.3	5.3					
8b W	" x $\frac{1}{4}$	6.3	5.6	4.9	4.2	3.5					

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 100.

Safe loads are in accordance with the working stresses prescribed by the London County Council (General Powers) Act, 1900, for stanchions of mild steel having "both ends fixed."

For other conditions and formulae, see notes commencing page 192 L.

Safe loads printed in italics are for heights greater than 40D.

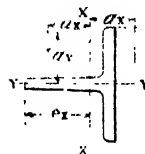
For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## STANCHIONS (or STRUTS).

### Steel Tees.

#### Dimensions and Properties.



Size, B x D x t inches.	Weight per foot in lbs.	Area in square inches.	Distance e <sub>x</sub> inches.	Radii of Gyration		Eccentricity Coefficients.	
				Axis Y-Y	Axis X-X	Axis Y-Y	Axis X-X
4 x 4 x ½	12.78	3.758	2.84	0.83	1.20	1 + 2.90a <sub>v</sub>	1 + 1.98a <sub>x</sub>
" x ¾	9.77	2.872	2.89	0.81	1.21	1 + 3.02a <sub>v</sub>	1 x 1.98a <sub>x</sub>
4 x 3 x ½	11.08	3.260	2.18	0.89	0.85	1 + 2.51a <sub>v</sub>	1 + 3.01a <sub>x</sub>
" x ¾	8.49	2.498	2.23	0.87	0.86	1 + 2.61a <sub>v</sub>	1 + 3.00a <sub>x</sub>
3½ x 3½ x ½	11.08	3.258	2.46	0.78	1.04	1 + 3.26a <sub>v</sub>	1 + 2.27a <sub>v</sub>
" x ¾	8.49	2.496	2.51	0.71	1.05	1 + 3.41a <sub>v</sub>	1 + 2.27a <sub>x</sub>
3 x 3 x ½	9.38	2.760	2.08	0.63	0.88	1 + 3.71a <sub>v</sub>	1 + 2.65a <sub>x</sub>
" x ¾	7.21	2.121	2.13	0.62	0.89	1 + 3.90a <sub>v</sub>	1 + 2.65a <sub>x</sub>
2½ x 2½ x ¾	5.92	1.741	1.75	0.52	0.74	1 + 4.61a <sub>v</sub>	1 + 3.17a <sub>x</sub>
" x ½	4.07	1.197	1.80	0.50	0.75	1 + 4.96a <sub>v</sub>	1 + 3.19a <sub>x</sub>

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

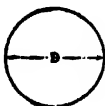
Each weight per foot is for the shaft only. Weight of connections, &c., to be added.

Least radii of gyration and relative eccentricity coefficients are printed in prominent type.  
W<sub>e</sub> = actual eccentric load; K = relative eccentricity coefficient; W<sub>c</sub> = equivalent concentric value; W<sub>c</sub> = W x K.

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for a<sub>v</sub> and a<sub>x</sub> respectively.

For full explanations of tables, see pages commencing page 192 L.

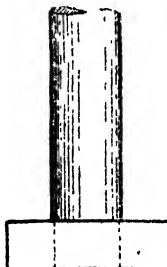
REDPATH, BROWN & CO., LIMITED.



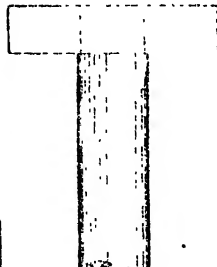
**STANCHIONS (or COLUMNS).**  
**Solid Round Steel.**

Safe Concentric Loads, in Tons.  
Ends Fixed.

Reference Mark.	Size, D inches.	HEIGHTS IN FEET.													
		6	8	10	12	14	16	18	20	22	24	26	28	30	
23 X	12	667	644	622	599	576	554	531	508	486	463	441	418	395	
22 X	11½	610	588	566	545	523	501	480	458	436	415	393	371	349	
21 X	11	555	534	511	493	472	451	431	410	389	368	348	327	306	
20 X	10½	503	483	463	444	424	404	384	364	345	325	305	285	265	
19 X	10	453	435	416	397	378	359	340	322	303	284	265	246	227	
18 X	9½	407	389	371	353	335	317	299	281	263	245	227	210	192	
17 X	9	362	345	328	311	294	277	260	243	226	209	192	176	159	
16 X	8½	320	304	288	272	256	240	224	208	192	176	160	144		
15 X	8	281	266	251	236	221	206	191	175	160	145	130			
14 X	7½	244	230	216	202	188	174	159	145	131	117				



BASE.



CAP.

Bases and Caps are formed of heavy steel slabs, bored out and shrunk on to the accurately machined column ends.

The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 100.

Safe loads are in accordance with the working stresses prescribed by the London County Council (General Powers) Act, 1909, for stanchions of mild steel, having "both ends fixed."

For other conditions and formulae, see notes commencing page 192 L.

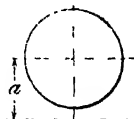
For explanations of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## STANCHIONS (or COLUMNS).

### Solid Round Steel.

#### Dimensions and Properties.



Diameter in inches.	Weight per foot in lbs.	Area in square inches.	Radius of Gyrations.	Eccentricity Coefficients.	
				For Semi-diameter.	General.
12	384.6	113.100	3.000	5	1+0.67 <i>a</i>
11½	353.2	103.870	2.875	5	1+0.70 <i>a</i>
11	323.2	95.033	2.750	5	1+0.73 <i>a</i>
10½	294.5	86.596	2.625	5	1+0.76 <i>a</i>
10	267.1	78.540	2.500	5	1+0.80 <i>a</i>
9½	241.0	70.882	2.375	5	1+0.84 <i>a</i>
9	216.3	63.617	2.250	5	1+0.89 <i>a</i>
8½	193.0	56.745	2.125	5	1+0.94 <i>a</i>
8	170.9	50.265	2.000	5	1+1.00 <i>a</i>
7½	150.3	44.179	1.875	5	1+1.07 <i>a</i>

SLABS OF THE UNDERNOTED WIDTHS AND THICKNESSES ARE STOCKED IN LENGTHS OF ABOUT 12 FEET.

Width and Thickness. Inches.	Suitable for Diameters.	Width and Thickness. Inches.	Suitable for Diameters.
18 x 4	10 to 8 inches	10 x 2	5 to 4 inches.
16 x 3½	8 to 7 "	9 x 1½	4 to 3½ "
14 x 3	7 to 6 "	8 x 1½	3½ to 2½ "
12 x 2½	6 to 6 "	—	—

Above sizes are for concentric loading.

Special calculations are necessary for the design of slab cap plates supporting eccentric loads. See Part IV.

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

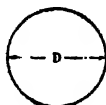
Each weight per foot is for the shaft only. Weight of base, &c., to be added.

*W*<sub>a</sub>=actual eccentric load; *K*=relative eccentricity coefficient; *W*<sub>c</sub>=equivalent concentric value; *W*<sub>c</sub>=*W*<sub>a</sub>×*K*.

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for *a*.

For full explanations of tables, see notes commencing page 192 L.

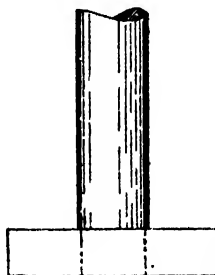
REDPATH, BROWN & CO., LIMITED.



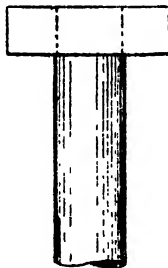
**STANCHIONS (or COLUMNS).**  
**Solid Round Steel.**

Safe Concentric Loads, in Tons.  
Ends Fixed.

Reference Mark.	Size, D Inches.	HEIGHTS IN FEET.														22
		4	6	8	10	11	12	13	14	15	16	18	20			
13 X	7	223	210	197	184	177	170	164	157	151	144	131	118	104		
12 X	6½	191	178	166	154	148	142	136	129	123	117	105	93·1			
11 X	6	161	149	138	127	121	115	110	104	98·9	93·3	81·9	70·6			
10 X	5½	133	123	112	102	97·4	92·2	87·0	81·8	76·6	71·4	61·1				
9 X	5	108	99·3	89·9	80·5	75·7	71·0	66·3	61·6	56·9	52·2					
8 X	4½	86·4	77·9	69·4	60·9	56·7	52·4	48·2	44·0	39·7						
7 X	4	66·6	59·0	51·5	43·9	40·2	36·4	32·6								
6 X	3½	49·3	42·7	36·1	29·5	26·2										
5 X	3	34·6	28·9	23·3	17·6											
4 X	2½	22·4	17·7	13·0												



BASE.



CAP.

Bases and Caps are formed of heavy steel slabs, bored out and shrunk on to the accurately machined column ends.

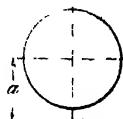
The above safe loads are tabulated for ratios of slenderness up to, but not exceeding 160.  
Safe loads are in accordance with the working stresses prescribed by the London County Council (General Powers) Act, 1909, for stanchions of mild steel having "both ends fixed."  
For other conditions and formulae, see notes commencing page 192 L.  
For explanation of properties, &c., see Part IV.

# REDPATH, BROWN & CO., LIMITED.

## STANCHIONS (or COLUMNS).

### Solid Round Steel.

#### Dimensions and Properties.



Diameter in inches.	Weight per foot in lbs.	Area in square inches.	Radius of Gyration.	Eccentricity Coefficients.	
				For Semi-diameter.	General.
7	130.9	38.485	1.750	5	1+1.15 <i>a</i>
6½	112.9	33.183	1.625	5	1+1.23 <i>a</i>
6	96.13	28.274	1.500	5	1+1.34 <i>a</i>
5½	80.78	23.758	1.375	5	1+1.46 <i>a</i>
5	66.76	19.635	1.250	5	1+1.60 <i>a</i>
4½	54.07	15.904	1.125	5	1+1.78 <i>a</i>
4	42.72	12.566	1.000	5	1+2.00 <i>a</i>
3½	32.71	9.621	0.875	5	1+2.29 <i>a</i>
3	24.03	7.069	0.750	5	1+2.67 <i>a</i>
2½	16.69	4.909	0.625	5	1+3.20 <i>a</i>

SLABS OF THE UNDERNOTED WIDTHS AND THICKNESSES ARE STOCKED IN LENGTHS OF ABOUT 12 FEET.

Width and Thickness. Inches.	Suitable for Diameters.	Width and Thickness. Inches.	Suitable for Diameters.
18 x 4	10 to 8 inches.	10 x 2	5 to 4 inches
16 x 3½	8 to 7 "	9 x 1½	4 " 3½ "
14 x 3	7 to 6 "	8 x 1½	3½ " 2½ "
12 x 2½	6 to 5 "	—	—

Above sizes are for concentric loading.

Special calculations are necessary for the design of slab cap plates supporting eccentric loads. See Part IV.

In each case the weight per foot given is the minimum that can be rolled, and a rolling margin of 2½ per cent. over this must be allowed. See page 7.

Each weight per foot is for the shaft only. Weight of base, &c., to be added.

We=actual eccentric load; K=relative eccentricity coefficient; Wc=equivalent concentric value; Wc=Wc x K.

In axial eccentricity coefficients substitute actual value of "arm of eccentricity" for *a*.

For full explanations of tables, see notes commencing page 192 L.

REDPATH, BROWN & CO., LIMITED.

**PART VI.—LONDON.**

**Explanations of the Tables.**

Pages 122L to 191L inclusive.

See Part IV. for general formulæ, explanations of properties, &c.

**Part VI.** All the tables in this part relate to simple and compound sections as stanchions, struts or columns.

— These are compiled in accordance with the Amendment of London Building Acts, London County Council (General Powers) Act, 1909, Part IV., with respect to buildings of Steel Skeleton Construction in London, for text of which see page 211 L.

**Arrangement.** The arrangement is identical with Part II., also relating to stanchions, which complies with the principal London Building Acts, 1894 to 1908, and Provincial Building requirements.

**Page Numbers.** For convenience of reference the pages of this part bear the same numbers as those of Part II., with the addition of the letter "L" signifying "London."

**Compound Stanchions.** A full range of plate thicknesses is given for each joist and channel compound stanchion.

In a series of superimposed stanchions it is convenient and economical to retain the same section of joist or channel throughout, varying the plate areas, only, in accordance with the loads.

The tables afford a ready means of selecting suitable types for this purpose.

## REDPATH, BROWN & CO., LIMITED.

The tabulated safe loads for each latticed stanchion, **Latticed Stanchions.** assume efficient bracing between the individual members composing the shaft.

Conventional *minimum* proportions of lattice bars and batten plates for concentric loading are indicated on the tables. Practical considerations will frequently cause the minimum proportions (especially of batten plates) to be increased considerably.

The conventional minimum proportions are not applicable to stanchions under "intentionally" eccentric loading.

Certain formulæ for the design of lattice bars are noticed in Part IV.

In structural steelwork applied to buildings, angle and tee stanchions or struts are usually the compression members of lattice girders or roof trusses. **Angle and Tee Stanchions or Struts.**

The tabulated safe loads for the condition of "both ends fixed" are generally applicable to such members, unless each end connection consists of one bolt or one rivet only.

In the latter case refer to the condition of "hinged ends," page 199L.

Solid round steel stanchions or columns are most useful **Solid Rounds.** in positions where considerations of space are of primary importance. For a given load the possible minimum of overall dimensions is attainable with this type.

Particular care should be taken to ensure *concentric* loading on solid round steel stanchions as the effect of *eccentricity* is relatively very great.

# REDPATH, BROWN & CO., LIMITED.

<b>Details.</b>	Various types of stanchions with suitable designs for bases, caps and connections are illustrated in Part V.
<b>Dimensions and Properties.</b>	All dimensions are stated in inches and all properties in inch units.
<b>Overall Sizes.</b>	D = depth, B = breadth, and t = thickness.
<b>Composition.</b>	The composition of compound stanchions is described in the first columns of the right-hand pages in the same manner as in Part I.
<b>Plate Thicknesses.</b>	When the plating on each flange exceeds $\frac{3}{4}$ of an inch, two or more plates may be used to form the total thickness required.
<b>Rivet Pitch.</b>	The standard rivet pitch for compound stanchions is 6 inches, the diameter being $\frac{3}{8}$ -inch or $\frac{1}{2}$ -inch as indicated on the tables.
<b>Weights per foot.</b>	<p>Each weight per foot in lbs. of compound plated stanchions and of double angles back to back, includes an allowance for rivet heads at standard pitch.</p> <p>Each weight per foot in lbs. is that of the plain or riveted shaft only. The weight of base, cap, connections, lattice bracing, batten plates, extra rivets, &amp;c., requires to be added in estimating the total weight of a complete stanchion.</p>
<b>Areas.</b>	Each area in square inches is the superficial area of a cross section at right angles to the longitudinal axis.
<b>Radii of Gyration.</b>	Least and greatest radii of gyration are tabulated for each stanchion.

# REDPATH, BROWN & CO., LIMITED.

The least radius of gyration is invariably printed in **Radii of prominent type. Gyration.**

For each joist and channel and stanchions compounded of these—excepting No. 24 L, page 137 L, and Nos. 32 M to 29 M inclusive, page 149 L—the least radius of gyration is about “Axis Y—Y” passing through the centre of gravity of the figure and parallel to the web or webs.

The greatest radius of gyration for each of these sections is about “Axis X—X” passing through the centre of gravity of the figure and parallel to the flanges.

The converse applies to Nos. 24 L and 32 M to 29 M, noted above.

The tabulated radii of gyration for each tee and tee shaped stanchion formed of two angles back to back, are also about central axes, but the least radius may be about “Axis Y—Y” parallel to the stalk, or about “Axis X—X” parallel to the table, depending upon the dimensions of the section.

For each simple or latticed angle stanchion, the least radius of gyration is about the major “Axis V—V” of the inertia ellipse. The greatest radius for each of these sections is about the minor “Axis U—U” of the inertia ellipse.

For solid round steel stanchions, all radii of gyration about central axes are identical.

No deduction is made for rivet holes in the calculation **Rivet Holes.** of radii of gyration of compound sections.

REDPATH, BROWN & CO., LIMITED.

**Tabular Loads.** The tabulated safe loads are without exception relative to the least radius gyration.

**Dimension "d."** Dimension "d" in the tables of compound stanchions, pages 136 L to 149 L, and pages 154 L to 159 L, is the spacing of the component joists or channels upon which the tabulated properties are based.

Any increase or decrease of "d" will therefore increase or decrease the radius of gyration about "Axis Y—Y," and, with the exception of No. 24 L, page 137 L, and Nos. 32 M to 29 M, page 149 L, will also increase or decrease the tabulated safe loads.

The maximum increase of safe load is reached when "Axis X--X" becomes the axis of least radius, safe loads relative to this axis being constant for all values of "d."

The tabulated safe loads for Nos. 24 L and Nos. 32 M to 29 M are the maximum for these sections, and will be decreased when "Axis Y—Y" becomes the axis of least radius.

**Concentric Loading.**

Each tabular load is described as "safe concentric."

This implies that the centre of application of the load or system of loading is, so far as practically possible, coincident with the central vertical axis of the stanchion, or in other words that there is no intentional eccentricity of loading.

**Ratio of Slenderness.**

If the height of a stanchion is divided by its least radius of gyration in the same unit dimension (both generally expressed in inches) the quotient is termed the "ratio of slenderness."

# REDPATH, BROWN & CO., LIMITED.

$l$  = height of stanchion in inches.

Ratio of  
Slenderness.

$k$  = least radius of gyration in inches.

$\frac{l}{k}$  = ratio of slenderness.

In the tables the nearest even height of a stanchion not exceeding that for which the "ratio of slenderness" is equal to 160 is taken as the limiting height for which a safe load is given.

Limiting  
Heights.

Some authorities prefer to limit the height of a stanchion to the lesser of the two values :—

- (1). 160 times the least radius of gyration.
- (2). 40 times the least overall dimension D. or B.

Frequently limit (2) gives a lower height than limit (1).

For this reason, safe loads on all heights greater than the limiting height by (2) are printed in italics in the tables.

Italics.

The tabulated safe loads have been calculated from the working stresses in tons per square inch, specified in the 1909 Amendment London Building Acts, for pillars of mild steel having "both ends fixed."

Tabulated  
Safe Loads.

For these working stresses and other end conditions, see page 201 L.

The following straight line formulæ are derived from the tables of specified working stresses and may be used in preference to interpolation for intermediate or for other ratios of length to least radius of gyration not tabulated in the "Amendment."

Formulæ.

REDPATH, BROWN & CO., LIMITED.

MILD STEEL PILLARS.

Ratio of Length to Least Radius of Gyration.	Working Stress in Tons per square inch of section.	Condition of Ends.
0 - 100	$4.5 - \frac{1}{40k}$	Hinged Ends.
100 - 140	$4.5 - \left( 2.5 + \frac{1}{20} - 100 \right)$	
0 - 140	$5.5 - \frac{1}{40k}$	One End Hinged and One End Fixed.
140 - 180	$5.5 - \left( 3.5 + \frac{1}{20} - 140 \right)$	
0 - 160	$6.5 - \frac{1}{40k}$	Both Ends Fixed.
160 - 210	$6.5 - \left( 4.0 + \frac{1}{20} - 160 \right)$	

$l$  = height of pillar in inches

$k$  = least radius of gyration in inches.

"Hinged ends" and "one end hinged, one end fixed."

Safe loads for "hinged ends" and "one end hinged, one end fixed" may be got directly from the tabular loads as follows:—

$W$  = tabular safe load in tons for "both ends fixed."

$W_x$  = required safe load in tons for another condition of ends.

$A$  = tabular area of section in square inches.

$\frac{l}{k}$  = ratio of length to least radius of gyration, or ratio of slenderness.

# REDPATH, BROWN & CO., LIMITED.

For "hinged ends":—

$$\frac{1}{k} \text{ 0 to 100. } W_s = W - 2A.$$

"Hinged ends"  
and "one end  
hinged, one end  
fixed."

$$*\frac{1}{k} \text{ 100 to 140. } W_s = W - \left( 2A + \frac{\frac{1}{k} - 100}{40} \right)$$

For "one end hinged, one end fixed":—

$$\frac{1}{k} \text{ 0 to 140. } W_s = W - A.$$

$$*\frac{1}{k} \text{ 140 to 160. } W_s = W - \left( A + \frac{\frac{1}{k} - 140}{40} \right)$$

\*For these higher ratios it will generally be more convenient to multiply the area by the working stress.

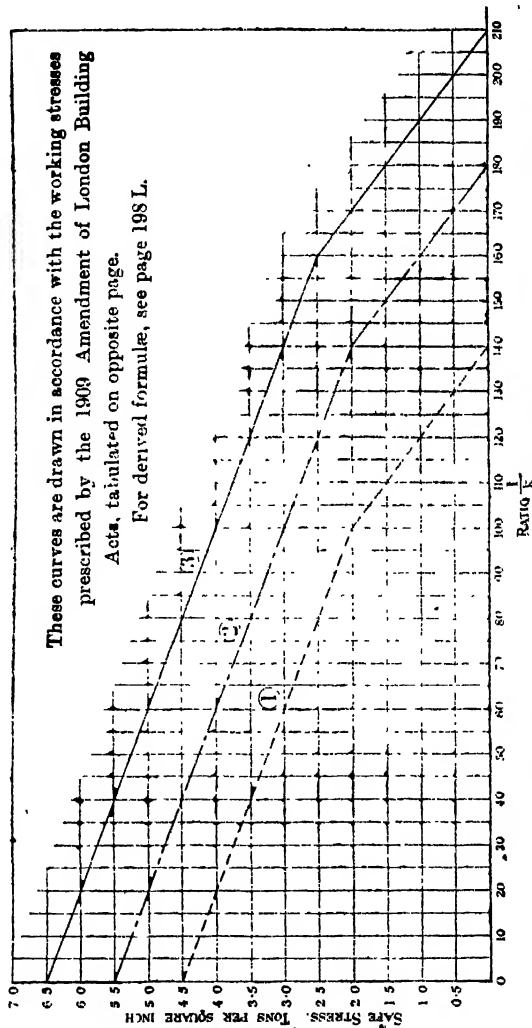
It may be pointed out that 100, 140 and 160 respectively are the economical limiting ratios of slenderness for the three conditions of ends, as above these values the working stresses are reduced 100 per cent. more rapidly than below them.

Economical  
Limiting Ratios.

In other words the reduction is at the rate of one ton per square inch for each increase of 40 of the ratios up to the foregoing limits, but at the rate of 2 tons per square inch for each similar increase beyond them.

REDPATH, BROWN & CO., LIMITED.

WORKING STRESSES IN TONS PER SQUARE INCH OF SECTION,  
BY 1909 AMENDMENT OF LONDON BUILDING ACTS,  
FOR STANCHIONS OF MILD STEEL UNDER CONCENTRIC LOADING.



- Curve (1). Stanchions with "Hinged Ends." .. Variation of Working Stresses for Ratios of  $\frac{L}{r}$  from 0 to 140.  
Curve (2). Stanchions with "One End Hinged" and "One End Fixed." .. Variation of Working Stresses for Ratios of  $\frac{L}{r}$  from 0 to 180.  
Curve (3). Stanchions with "Both Ends Fixed." .. Variation of Working Stresses for Ratios of  $\frac{L}{r}$  from 0 to 210.

## REDPATH, BROWN &amp; CO., LIMITED.

Working Stresses in tons, per square inch of Section, for Stanchions of Mild Steel under Concentric Loading in accordance with the 1909 Amendment of London Building Acts.

$\frac{1}{k}$	(1).	(2).	(3).	$\frac{1}{k}$	(1).	(2).	(3).	$\frac{1}{k}$	(1).	(2).	(3).
0	4.5	5.5	6.5	72	2.7	2.7	4.7	142	..	1.9	2.96
4	4.4	5.4	6.4	74	2.65	2.65	4.65	144	..	1.8	2.9
8	4.35	5.35	6.35	76	2.6	2.6	4.6	146	..	1.7	2.86
12	4.3	5.3	6.3	78	2.55	2.55	4.55	148	..	1.6	2.8
16	4.25	5.25	6.25	80	2.5	2.5	4.5	150	..	1.5	2.75
20	4.2	5.2	6.2	82	2.45	2.45	4.45	152	..	1.4	2.7
24	4.15	5.15	6.15	84	2.4	2.4	4.4	154	..	1.3	2.65
28	4.1	5.1	6.1	86	2.35	2.35	4.35	156	..	1.2	2.6
32	4.05	5.05	6.05	88	2.3	2.3	4.3	158	..	1.1	2.55
36	4.0	5.0	6.0	90	2.25	2.25	4.25	160	..	1.0	2.5
40	3.95	4.95	5.95	92	2.2	2.2	4.2	162	..	0.9	2.4
44	3.9	4.9	5.9	94	2.15	2.15	4.15	164	..	0.8	2.3
48	3.85	4.85	5.85	96	2.1	2.1	4.1	166	..	0.7	2.2
52	3.8	4.8	5.8	98	2.05	2.05	4.05	168	..	0.6	2.1
56	3.75	4.75	5.75	100	2.0	2.0	4.0	170	..	0.5	2.0
60	3.7	4.7	5.7	102	1.9	2.95	3.95	172	..	0.4	1.9
64	3.65	4.65	5.65	104	1.8	2.9	3.9	174	..	0.3	1.8
68	3.6	4.6	5.6	106	1.7	2.85	3.85	176	..	0.2	1.7
72	3.55	4.55	5.55	108	1.6	2.8	3.8	178	..	0.1	1.6
76	3.5	4.5	5.5	110	1.5	2.75	3.75	180	..	0.0	1.5
80	3.45	4.45	5.45	112	1.4	2.7	3.7	182	..	..	1.4
84	3.4	4.4	5.4	114	1.3	2.65	3.65	184	..	..	1.3
88	3.35	4.35	5.35	116	1.2	2.6	3.6	186	..	..	1.2
92	3.3	4.3	5.3	118	1.1	2.55	3.55	188	..	..	1.1
96	3.25	4.25	5.25	120	1.0	2.5	3.5	190	..	..	1.0
100	3.2	4.2	5.2	122	0.9	2.45	3.45	192	..	..	0.9
104	3.15	4.15	5.15	124	0.8	2.4	3.4	194	..	..	0.8
108	3.1	4.1	5.1	126	0.7	2.35	3.35	196	..	..	0.7
112	3.05	4.05	5.05	128	0.6	2.3	3.3	198	..	..	0.6
116	3.0	4.0	5.0	130	0.5	2.25	3.25	200	..	..	0.5
120	2.95	3.95	4.95	132	0.4	2.2	3.2	202	..	..	0.4
124	2.9	3.9	4.9	134	0.3	2.15	3.15	204	..	..	0.3
128	2.85	3.85	4.85	136	0.2	2.1	3.1	206	..	..	0.2
132	2.8	3.8	4.8	138	0.1	2.05	3.05	208	..	..	0.1
136	2.75	3.75	4.75	140	0.0	2.0	3.0	210	..	..	0.0

(1) = Working stresses for "hinged ends."

(2) = " " " " "one end hinged, one end fixed."

(3) = " " " " "both ends fixed."

The working stresses for ratios of slenderness beyond the economical limits (see page 199 L) are printed in italics.

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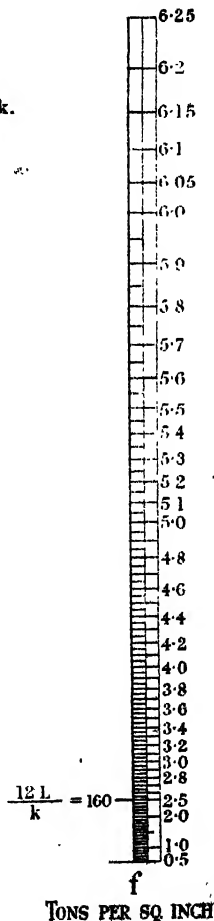
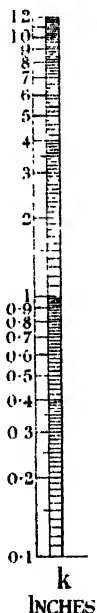
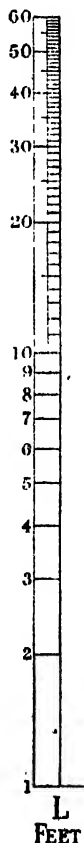
Stanchions.

L.C.C. (1909) Amendment

## ALIGNMENT CHART I.

Lay transparent straight edge across scales—

- (a) At height in feet on scale L.
- (b) " radius of gyration on scale k.
- (c) Read safe stress on scale f.



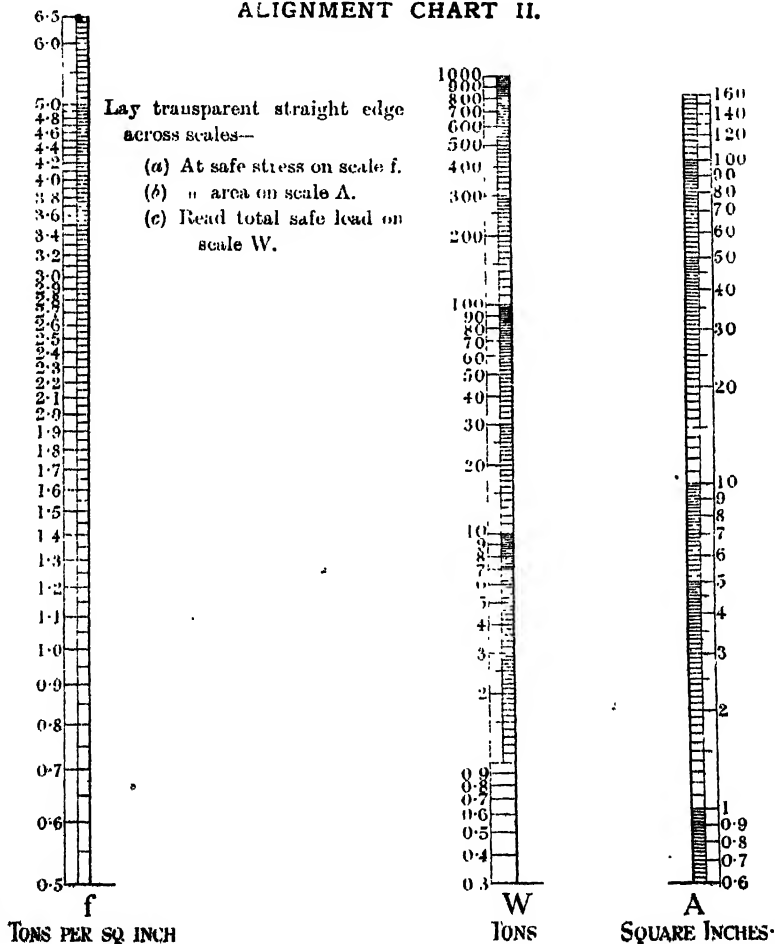
$$\frac{12 L}{k} = 160$$

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L.C.C. (1905) Amendment.

Stanchions.

## ALIGNMENT CHART II.



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**Eccentric Loading.**

Eccentric loading is of two descriptions, viz.:—  
“Accidental” and “Intentional”

**Accidental Eccentricity.**

“Accidental eccentricity” is due to the fact that the physical and geometrical axes of the practical column rarely coincide throughout the height, owing to the inherent but practically unobservable defects of material and workmanship.

The theory of the strength of columns is based on the primary assumption of a perfectly centred column of perfect material and workmanship, the factor of safety applied to the derived formulæ for ultimate strength being sufficient to cover the necessary allowance, *inter alia*, for “accidental eccentricity” thus giving suitable safe working stresses for concentric loading.

**Intentional Eccentricity.**

“Intentional eccentricity” occurs when the perpendicular distance from the centre of application of a load or system of loading to either or both principal axes of the stanchion is a quantity measurable by ordinary practical methods.

**Eccentricity (general).**

In these notes by “eccentricity” will now be understood “intentional eccentricity” as “accidental eccentricity” is not considered further.

**Arm of Eccentricity.**

The measurable distance referred to above is termed the “arm of eccentricity,” and is expressed in inches.

**Principal Axes.**

The “principal axes” are the “axis of least radius” and “axis of greatest radius.”

**Loading.**

Loading is said to be “eccentric about the axis” to which the “arm of eccentricity” is perpendicular.

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The tabular eccentricity coefficients are derived from the general formulæ for eccentric loading, for which see Part IV. Eccentric Loading.

For each stanchion, eccentricity coefficients relative to both of the "principal axes" of the section are given.

The eccentricity coefficients relative to the "axis of least radius" are printed in prominent type in the tables. Prominent Type.

The coefficients in the tables under headings "Axis Y—Y," "Axis X—X," "Axis V—V," and "Axis U—U" are respectively relative to these "principal axes," and may be termed "axial coefficients." Axial Coefficients.

To complete the "axial coefficients" it is only necessary to substitute for  $U$ ,  $A$ ,  $A$ ,  $A$ , the actual value in inches of the "arm of eccentricity."

The "axial coefficients" are of general application for any degree of eccentricity, care being taken to select the coefficient having the same reference letters as the axis about which the loading is eccentric.

Special note may be made of the "axial coefficients" for each channel stanchion, pages 150 L to 153 L. Channel Coefficients.

As "axis Y—Y" for this type is not an axis of symmetry, it is necessary to consider on which side of this axis the eccentric loading is placed.

When the "arm of eccentricity" is measured in the same direction as dimension  $e$ , use coefficient "Axis Y—Y  $e$ ," and, conversely, when the centre of application of the load is on the other side of "Axis Y—Y" use coefficient "Axis Y—Y  $C$ ."

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## Angle and Tee Coefficients.

For each angle, tee and tee-shaped stanchion the "axial coefficients" relative to the asymmetrical axes V—V, U—U, and X—X, take into account the perpendicular distance  $C_v$ ,  $C_u$ , or  $C_x$  to the extreme fibre of the section, irrespective of the side of the axis on which the loading occurs. The worst case is thus provided for.

## Web and Flange Coefficients.

In addition to the "axial coefficients" for each joist and channel and for each stanchion compounded of either of these sections there are two coefficients for special conditions of eccentric loading, viz.:—"Web" and "Flange" respectively relative to "Axis Y—Y" and "Axis X—X."

These are applicable when the "arm of eccentricity" is identical with the perpendicular distance from "Axis Y—Y" or "Axis X—X" to the outer surface of the web or flange respectively.

This is usually taken to be the case in good construction when the eccentric load is transmitted by a girder properly connected to a side of a stanchion.

## Equivalent Concentric Load.

By the use of the eccentricity coefficient for the axis about which a load or system of loading is eccentric an "equivalent concentric value" of the load relative to that axis may be obtained.

Let  $W_e$  = actual eccentric load in tons.

$K$  = eccentricity coefficient for the axis about which " $W_e$ " is eccentric.

$W_c$  = equivalent concentric load value in tons for that axis.

Then  $W_c = W_e \times K$ .

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It follows from the above that if a tabular concentric load is divided by the eccentricity coefficient printed in prominent type, the maximum safe eccentric load relative to the "axis of least radius" of the section is obtained directly.

**Safe  
Eccentric Load.**

The following examples illustrate the application of the tables to the design of eccentrically loaded stanchions, and also the use of the Alignment Charts, pages 202 L and 203 L.

**Examples.**

**(A) Loading eccentric about "axis of least radius" only.**

*Example 1.*—A stanchion 16 feet high supports an eccentric load of 58 tons, transmitted directly to its web surface by a girder.

Required a suitable section.

Select No. 19 J, page 122 L, steel joist 10 × 8 which will support a safe concentric load of 80·8 tons on 16 feet.

Multiply 58 tons, the actual concentric load, by 1·35 the "web" eccentricity coefficient for the section.

The product 78·3 tons is the equivalent concentric load value, therefore the selected stanchion is suitable.

*Example 2.*—A stanchion 20 feet high supports a system of eccentric loading amounting to 160 tons, the "arm of eccentricity" about the "axis of least radius" being 2 inches.

Required a suitable section.

Select No. 144 M, page 144 L, composed of two steel joists 14 × 6b, and four flange plates 14" ×  $\frac{1}{2}$ " which will support a safe concentric load of 303 tons on 20 feet.

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## Example.

Substitute 2 ins., the given "arm of eccentricity," for  $a_r$  and obtain  $(1 + 0.41 \times 2) = 1.82$  as the "Axis Y—Y," or "axis of least radius" eccentricity coefficient.

Multiply 160 tons, the actual eccentric load, by 1.82.

The product 291 tons is the equivalent concentric load value, therefore the selected stanchion is suitable.

**(B) Loading eccentric about "axis of greatest radius" only.**

*Example 3.*—A stanchion 12 feet high supports an eccentric load of 70 tons transmitted directly to its flange surface by a girder.

Required a suitable section.

Select No. 31 P, page 156 L, composed of two steel channels  $10 \times 3\frac{1}{2}$  and two flange plates  $12" \times \frac{5}{8}"$ .

Note height 12 feet; area, 31.6 square inches, and greatest radius of gyration "Axis X—X" 4.57 inches.

Transfer these values to the Alignment Charts, pages 202 L and 203 L.

On Chart I. lay a straight edge across the three vertical scales at the height of 12 feet on scale L, at the radius of gyration 4.57 inches on scale k, and read safe stress as 5.71 tons per square inch on scale f.

On Chart II. lay straight edge at 5.71 tons on scale f, at the area 31.6 square inches on scale A and read safe concentric load for "Axis X—X" as 180 tons on scale W.

Divide 180 tons by 2.52 the flange eccentricity coefficient for the section.

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The quotient 71.4 tons is the safe flange eccentric load, *Examples.* therefore the selected stanchion is suitable.

*Example 4.*—A stanchion 14 feet high supports a system of eccentric loading amounting to 265 tons the “arm of eccentricity” about the “axis of greatest radius” being  $1\frac{1}{2}$  inches.

Required a suitable section.

Select No. 254 K, page 126 L, composed of one steel joist  $20 \times 7\frac{1}{2}$  and four flange plates  $14'' \times \frac{1}{2}''$ .

Note height 14 feet; area, 54.1 square inches and greatest radius of gyration “Axis X—X” 9.37 inches.

Transfer these values to the Alignment Charts, pages 202 L and 203 L.

On Chart I. by the method described read 6 tons per square inch on scale f.

On Chart II. read safe eccentric load for “Axis X—X” as 324 tons on scale W.

Substitute 1.5 the “arm of eccentricity” for  $a_x$  and obtain  $(1 + 0.13 \times 1.5) = 1.195$  as the “Axis X—X” or “axis of greatest radius” eccentricity coefficient.

Divide 324 tons by 1.195.

The quotient 270 tons is the safe load for an eccentricity of  $1\frac{1}{2}$  inches about “Axis X—X,” therefore the selected stanchion is suitable.

### (C) Loading eccentric about both axes.

Select a stanchion from the tables as in Examples 1 and 2 as if the loading were eccentric about the “axis of

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<b>Examples.</b>	least radius" only ; then by use of the Alignment Charts and eccentricity coefficient for the "axis of greatest radius" check the section for the load eccentric about the "axis of greatest radius."
<b>Combined Loading.</b>	If a stanchion supports concentric in addition to eccentric loading, the former, if treated separately, must be added to the equivalent concentric load value to give the total equivalent concentric load.
<b>Maximum Load.</b>	The actual load eccentric or concentric for the "axis of greatest radius" must in no case exceed the tabular load—calculated for the "axis of least radius."

For notes on the location of the "centre of application" of load systems, see Part IV.

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Act, 1909.**

**PART IV.**

**AMENDMENT OF LONDON BUILDING ACTS.**

20. In this Part of this Act the expression "the principal Acts" means the London Building Acts 1894 to 1908.

Definition of  
principal  
Acts.

21. Words and expressions used in this Part of this Act shall unless the context otherwise requires bear the meanings assigned to them in the principal Acts and those Acts and this Part of this Act may be cited together as the London Building Acts 1894 to 1909.

Interpretation  
and effect of  
this Part of A.

For the purposes of this Part of this Act the expression "pillar" shall unless otherwise stated mean a metal pillar and shall include all columns and stanchions or an assemblage of columns or stanchions properly riveted or bolted together and the expression "girder" shall mean a metal girder or joist and the expression "tribunal of appeal" means the tribunal of appeal as constituted by this Part of this Act.

22. Notwithstanding anything contained in the principal Acts requiring buildings to be enclosed with walls of the thicknesses and of the materials therein respectively described it shall be lawful to erect subject to the provisions of this section buildings wherein the loads and stresses are transmitted through each storey to the foundations by a skeleton framework of metal or partly by a skeleton framework of metal and partly by a party wall or party walls but buildings so erected shall

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respect to  
buildings of  
iron and steel  
skeleton  
construction.

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buildings, etc.

(subject to any exemptions contained in the principal Acts or any of them) be subject to and comply with all such provisions of the principal Acts or any of them and any byelaws in force thereunder as may not be inconsistent with or contrary to the provisions of this section. The following are the provisions which shall apply in respect of the construction of the skeleton framework and the foundations walls floors staircases and other parts of the structure of such buildings:—

- (1) All rolled steel used in such construction shall comply with the British standard specification for structural steel for bridges and general building construction from time to time in operation and every pillar or girder shall be of iron or steel or any other structural metal which may hereafter be standardised by the Engineering Standards Committee:
- (2) The skeleton framework of a building together with the party wall or party walls (if any) upon which such framework bears shall be capable of safely and independently sustaining the whole dead load and the superimposed load bearing upon such framework and party wall or party walls:
- (3) All pillars in the external walls of a building shall be completely enclosed and protected from the action of fire by a casing of brickwork terra-cotta concrete stone tiles or other incombustible materials at least four inches thick the whole being properly bonded or secured together:
- (4) All girders in the external walls of a building shall be similarly enclosed and encased with brickwork.

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- terra-cotta concrete stone tiles or other incombustible materials at least four inches thick properly tied and bonded or secured to the adjoining work but the casing on the underside of such girders and to the edges of the flanges thereof and plates and angles connected therewith may be of any thickness not less than two inches :
- (5) All pillars and girders (other than pillars and girders in the external walls of a building) shall be protected from the action of fire by being encased to the satisfaction of the district surveyor and to a thickness of not less than two inches in brickwork terra-cotta concrete metal lathing and plaster or cement but the casing on the upper surface of the upper flange of all girders and on the lower surface of all subsidiary joists may be of any thickness not less than one inch. Wood firrings shall not be used in connexion with any such casing. Provided that this subsection shall not apply in the case of buildings of only one storey and not more than twenty-five feet in height :
- (6) Every girder shall be secured against buckling whenever the length of the girder exceeds thirty times the width of the compression flange and the web of every girder shall be secured against buckling in every case in which the depth of the web exceeds sixty times the thickness thereof :
- (7) The span of a girder shall not exceed twenty-four times the depth of the girder unless the calculated deflection of such girder is less than one four hundredth part of the span :
- Provisions with respect to buildings, etc.

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- (8) Wherever two or more girders are arranged alongside and closely adjacent to one another and are intended to act together they shall be fixed together by means of iron separators and bolts or by riveted plates or in any other equally efficient manner approved by the district surveyor. Separators or plates or other members acting as separators shall not be placed at a distance apart exceeding five times the depth of the girders to which they are attached and shall also be placed immediately over all supports and immediately under or at all concentrated loads :
- (9) All girders for supporting external walls shall be placed at the floor level of a storey or at a distance of not more than five feet above or below such floor level :
- (10) Rivets shall be used in all cases where reasonably practicable but where bolts are used they shall extend through the full thickness of the nuts attached thereto and the nuts shall in all cases be so secured as to avoid the risk of their becoming loose. The distance from the edge of a rivet hole or bolt hole to the edge of the plate bar or member shall not be less than the diameter of the rivet or bolt. Rivets shall be so placed that their centres shall not be closer together than three times the diameter of the rivets. The pitch of rivets shall be measured in a continuous straight line and such straight line pitch in girders pillars and roofwork shall not exceed sixteen times the thickness of the thinnest plate bar or member through which they pass :

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- (11)—(A) An external wall may be of any thickness not less than eight and a half inches for the topmost twenty feet of its height and thirteen inches for the remainder of its height below such topmost twenty feet. Provided that a less thickness shall be allowed in any case in which under the London Building Act 1894 such less thickness is prescribed but that nothing in this sub-section shall override any of the requirements of this section in regard to the thickness of casing in connexion with pillars and girders and that in any case in which an external wall or portion of an external wall is not supported or carried or secured by metal framework within the limits of height and length prescribed by the First Schedule to the London Building Act 1894 for the purpose of determining the thickness of walls such external wall or portion of external wall shall be of a thickness not less than that prescribed by such schedule :

Provisions with respect to buildings, etc.

(B) All party walls shall be of the thicknesses prescribed by the principal Acts :

(C) All brickwork and work in which terra-cotta concrete stones tiles or other similar materials are used shall be executed in Portland cement mortar and shall be bedded close up to the metal framework without any intervening cavity and all joints shall be made full and solid. The cement so used shall be in accordance with the British standard specification from time to time in operation. Provided that in party walls (exclusive of any parts thereof immediately surrounding metal

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framework) or other internal brickwork not constructed to carry loads or stresses provided for under this section lime mortar may be used in accordance with the provisions of the principal Acts and any bye-laws in force thereunder :

(12)—(A) No plate or bar in any steel or wrought iron pillar shall in any part be less than a quarter of an inch thick and the bases of all such pillars shall be at right angles to the axis :

(B) All joints in such pillars shall be close butted with cover-plates properly riveted and all joints between such pillars shall be properly fixed and made and unless unavoidable no joint shall be made between such pillars except at or as near as may be reasonably practicable to the level of a girder properly secured to such pillars :

(C) The foot of every such pillar shall have a proper base-plate riveted thereto with sufficient gusset pieces to distribute properly the load on the foundations and the gusset pieces shall have sufficient rivets to transmit the whole of the load on to the base-plates :

(D) Where any such pillars are built up hollow the cavities shall either be filled up with concrete or be covered in at both ends by metal plates riveted thereto :

(13)—(A) The width of every cast-iron pillar shall be not less than five inches and the metal of which such pillar is composed shall not be in any part of less thickness than three-quarters of an inch or

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one-twelfth of the least width of such pillar (which-  
ever shall be the greater):

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(b) The cap and base of every such pillar shall be in one piece with the pillar or be connected thereto with a properly turned and bored joint sufficiently fixed:

(c) The ends of all such pillars shall be at right angles to the axis:

(d) All joints between such pillars shall be at or as near as may be reasonably practicable to the level of a girder properly secured to such pillars and shall be fixed and made with not fewer than four bolts of not less diameter than the least thickness of metal in the pillar. If more than four bolts are used the diameter of the bolts may be reduced proportionately but no bolt shall be less than three-quarters of an inch in diameter:

(e) The base of every such pillar shall have such area as may be necessary to distribute properly the load on the foundations:

(14) The base of every pillar shall be properly bedded so as to transmit uniformly the load upon such pillar to the foundations:

(15) The stress in any metal interposed between the ends of a superimposed pillar and a pillar beneath shall not exceed the stress on the superimposed pillar and the least width across such interposed metal shall not be less than the least width of the superimposed pillar:

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- (16) All floors and staircases (together with their enclosing walls) shall be constructed throughout of fire-resisting materials and be carried upon supports of fire-resisting materials :
- (17) All structural metalwork comprised in the skeleton framework of a building shall be cleaned of all scale dust and loose rust and be thoroughly coated with one coat of boiled oil tar or paint before erection and after erection shall receive at least one additional coat. Where such metalwork is to be embedded or encased in brickwork terra-cotta concrete stone tiles or other incombustible materials one coat of Portland cement wash of adequate consistency applied after erection may be used in lieu of coats of oil tar or paint :
- (18)—(A) The dead load of a building shall consist of the actual weight of walls floors roofs partitions and all other permanent construction comprised in such building :
- (B) The superimposed load in respect of a building shall consist of all loads other than the dead load :
- (C) For the purpose of calculating the loads on foundations pillars (including brick pillars) piers walls framework girders and other constructions carrying loads in buildings the superimposed load on each floor and on the roof shall be estimated as equivalent to the following dead load :—

For a floor intended to be used wholly or principally for the purposes of human habitation or for domestic purposes seventy pounds per square foot of floor area;

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For a floor intended to be used wholly or principally for the purpose of an office or a counting-house or for any similar purpose one hundred pounds per square foot of floor area ;

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For a floor intended to be used wholly or principally for the purpose of a workshop or retail shop one hundred and twelve pounds per square foot of floor area ;

For every floor in a building of the warehouse class not intended to be used wholly or principally for any of the purposes aforesaid not less than two hundred and twenty-four pounds per square foot of floor area. In every building of the warehouse class a notice shall be exhibited in a conspicuous place on each storey of such building stating the maximum superimposed load per square foot which may be carried on the floor of such storey ;

For a roof the plane of which inclines upwards at a greater angle than twenty degrees with the horizontal the superimposed load (which shall for this purpose be deemed to include wind pressure) shall be estimated at twenty-eight pounds per square foot of sloping surface ;

For all other roofs the superimposed load shall be estimated at fifty-six pounds per square foot measured on a horizontal plane :

Provided that if the superimposed load on any floor or roof is to exceed that herein-before specified for such floor or roof such greater load

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shall be provided for pursuant to subsection (2) of this section :

Provided also that in the case of any floor intended to be used for a purpose for which a superimposed load is not specified in this subsection the superimposed load to be carried on such floor shall be provided for pursuant to the said subsection (2) :

- (19) For the purpose of calculating the total load to be carried on foundations pillars (including brick pillars) piers and walls in buildings of more than two storeys in height the superimposed loads for the roof and topmost storey shall be calculated in full in accordance with the last preceding subsection of this section but for the lower storeys a reduction of the superimposed loads shall be allowed as follows :—

For the storey next below the topmost storey a reduction of five per centum of the full superimposed load for such next storey calculated as aforesaid ;

For the next succeeding lower storey a reduction of ten per centum of the full superimposed load for such storey calculated as aforesaid and for each succeeding lower storey a further reduction of five per centum of the full superimposed load for each such storey calculated as aforesaid. Provided always that the total reduction in respect of any storey shall not exceed fifty per centum of the full superimposed load for such storey ;

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No such reduction as aforesaid shall be allowed Provisions with respect to buildings, etc.  
in the case of a building of the warehouse class :

(20) All buildings shall be so designed as to resist safely a wind pressure in any horizontal direction of not less than thirty pounds per square foot of the upper two-thirds of the surface of such buildings exposed to wind pressure :

(21)—(A) The working stresses on pillars of cast iron or mild steel due to the loads thereon (other than stresses induced by wind pressure) shall not exceed those specified in the two next following tables according to the several ratios therein specified or a proportionate load for intermediate or other ratios :—

### CAST-IRON PILLARS.

Ratio of Length to least Radius of Gyration.	Working Stresses in Tons per Square Inch of Net Section.		
	Hinged Ends.	One End hinged and one End fixed.	Both Ends fixed.
20	3.5	4.0	4.5
30	3.0	3.5	4.0
40	2.5	3.0	3.5
50	2.0	2.5	3.0
60	1.5	2.0	2.5
70	1.0	1.5	2.0
80	.5	1.0	1.5

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## MILD STEEL PILLARS.

Provisions with respect to buildings, etc.

Ratio of Length to least Radius of Gyration.	Working Stresses in Tons per Square Inch of Section.		
	Hinged Ends.	One End hinged and one End fixed.	Both Ends fixed.
20	4.0	5.0	6.0
40	3.5	4.5	5.5
60	3.0	4.0	5.0
80	2.5	3.5	4.5
100	2.0	3.0	4.0
120	1.0	2.5	3.5
140	0.0	2.0	3.0
160		1.0	2.5
180		0.0	1.5
200			0.5
210			0.0

(b) The working stresses on wrought-iron pillars due to the loads thereon (other than stresses induced by wind pressure) shall not exceed two-thirds of the stresses herein-before specified with respect to mild steel pillars:

(c) Any such pillar eccentrically loaded shall have the stresses caused by such eccentricity computed and the combined stresses resulting from such eccentricity at any part of such pillar when added to all other stresses at that part shall in no case

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- exceed the working stresses specified in this subsection : Provisions with respect to buildings, etc.

Provided that working stresses exceeding those specified in paragraphs (A) (B) and (c) of this subsection by not more than twenty-five per centum may be allowed in cases in which such excess is due to stresses induced by wind pressure :

(D) The eccentric load of a pillar shall be considered to be distributed uniformly over the area of the cross section of such pillar at the next lower level at which such pillar is fixed and secured in the direction of eccentricity :

- (22) The working stresses of iron and steel (except in the case of pillars as herein-before provided) shall not exceed the following :—

	Working Stresses in Tons per Square Inch.			
	Tension.	Compression.	Shearing.	Bearing.
Cast iron . .	1.5	8	1.5	10
Wrought iron .	5	5	4	7
Mild steel . .	7.5	7.5	5.5	11

- (23) In the case of any rivet used in double shear the working shear on such rivet shall not exceed one and three-quarter times the working shear allowed under this section on a like rivet when used in single shear :

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- (24) The pressures of foundations on the natural ground shall not exceed the following :—

—	Tons per Square Foot.
Natural bed of soft clay or wet or loose sand . . .	1
Natural bed of ordinary clay or confined sand . . .	2
Natural bed of compact gravel London blue clay or chalk	4

- (25) The pressure on concrete foundations shall not exceed twelve tons per square foot :

- (26) No disengaged brick pillar shall have a height without proper lateral supports of more than six times its least width but any such pillar with proper lateral supports may have a height between such supports not more than twelve times the least width of such pillar. Such width shall in no case be less than thirteen-and-a-half inches :

- (27) The pressure on any brickwork shall not exceed the following :—

—	Tons per Square Foot.
Blue brick in cement mortar . . . . .	12
Hard brick (including London stock) in cement mortar	8
Ordinary brick in cement mortar . . . . .	5

- (28) The Council may prescribe the materials and the proportions of the materials to be used in any concrete provided under the provisions of this

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section to which the provisions of the principal Acts or any of them or any byelaws in force thereunder do not apply :

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(29) It shall be lawful to make any addition to or alteration of or to do other work to in or upon a building in accordance with the provisions of this section provided that the loads and stresses in the part of a building so added or altered or to in or upon which such other work is done are transmitted from the roof to the foundations by a skeleton framework of metal or partly by a skeleton framework of metal and partly by a party wall or party walls and the provisions of this section shall in all respects apply to such part of a building as if the same were a separate building :

(30) Any structural metal hereafter standardised by the Engineering Standards Committee as before mentioned shall be used in the erection of buildings or additions alterations or other work made or done under the provisions of this section only subject to such terms and conditions as the Council may think fit to attach either generally or in any particular case to the use of such metal but any person dissatisfied with any term or condition attached by the Council may appeal to the tribunal of appeal. Any person failing to comply with any such term or condition attached by the Council or (in the event of appeal) by the tribunal of appeal shall be liable to a penalty to be recoverable in a summary manner not

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exceeding twenty pounds and to a daily penalty not exceeding the like amount :

(31) In the case of the erection of a new building of metal skeleton framework or the making of any addition or alteration or the carrying out of other work under the provisions of this section the notice required to be served on the district surveyor under section 145 of the London Building Act 1894 shall be accompanied (A) in the case of a new building by plans and sections of sufficient detail to show the construction thereof together with a copy of the calculations of the loads and stresses to be provided for and particulars of the materials to be used and should such plans sections calculations or particulars be in the opinion of the district surveyor not in sufficient detail the person depositing the same shall furnish the district surveyor with such further plans sections calculations or particulars as he may reasonably require and (B) in the case of an alteration or addition or other work as aforesaid by such plans sections calculations and particulars as the district surveyor may reasonably require :

(32) The district surveyor may for the purpose of due supervision of the construction of a building require to be furnished with reasonable proof as to the quality of metal to be used in such construction and may if not furnished with such proof or for any other reason require the builder or other person causing or directing the work to be executed to make any tests which the district surveyor may

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consider necessary and to drill any pillar (in all cases if reasonably practicable before the same is encased) at any point to ascertain its thickness :

Provisions with respect to buildings, etc.

(33) Any person dissatisfied with any requirement of the district surveyor under this section may within fourteen days of the date of the service of a notice from the district surveyor of such requirement appeal to a petty sessional court who may make an order affirming such requirement or otherwise and every builder or other person ~~failing~~ to comply with any such order shall be liable to a penalty (to be recoverable in a summary manner) not exceeding twenty pounds a day during every day of the continuance of the non-compliance with such order :

(34) In order to facilitate the erection of buildings of metal skeleton framework it shall be lawful for the Council to modify or waive any of the requirements of sub-sections (3) (4) (5) (8) (9) (11) (12) (B) (17) (20) (24) and (25) of this section upon and subject to such terms and conditions as they may think fit and any person dissatisfied with the refusal of the Council to modify or waive any of such requirements or with any term or condition which the Council may attach to any modification or waiver may appeal to the tribunal of appeal. Any person failing to comply with any term or condition attached by the Council or (in the event of appeal) by the tribunal of appeal to such modification or waiver shall be liable to a penalty to be recoverable in a summary manner not

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exceeding twenty pounds and to a daily penalty not exceeding the like amount :

- (35) If the Council within the period of one month or in the event of such period of one month commencing or expiring on any day between the eighth day of August and the fourteenth day of September (both inclusive) then within a period of two months after the receipt of a written application for the modification or waiver of any of the requirements of this section which the Council are empowered to waive or modify fail to give notice to the applicant of their refusal or grant thereof the Council shall be deemed to have granted such application.

Power to make regulations as to use of reinforced concrete.

23.—(1) The Council may make regulations with respect to the construction of buildings wholly or partly of reinforced concrete and with respect to the use and composition of reinforced concrete in such construction and for the purpose of framing such regulations may carry out such investigations and make such tests as they may deem necessary and the provisions of this section and of any such regulations shall (subject to any exemptions contained in the principal Acts or any of them) have effect notwithstanding any provisions of the said Acts or any of them or any byelaw in force thereunder which may be inconsistent therewith or contrary thereto.

(2) Subject to such regulations as aforesaid buildings may be constructed wholly or partly of reinforced concrete but except as provided by this section or by such regulations buildings so constructed shall (subject to any exemptions contained in the principal Acts or any of them)

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be subject to and comply with all such provisions of the said Acts or any of them and of any byelaws in force thereunder as may not be inconsistent with or contrary to the provisions of this section or any regulations in force thereunder.

Power to make regulations as to use of reinforced concrete.

(3) No such regulations shall have any force or effect unless or until they shall have been submitted to and confirmed at a meeting of the Council subsequent to that at which the regulations shall have been made nor shall any such regulations have any force or effect until the same shall have been allowed by the Local Government Board.

(4) The Council shall give to the Surveyors' Institution the Institution of Civil Engineers the Royal Institute of British Architects and the Concrete Institute notice of their intention to apply to the Local Government Board for allowance of any regulations made under this section.

(5) All regulations made and confirmed and allowed as aforesaid shall be published in the London Gazette and printed and hung up at the County Hall and be open to public inspection without payment and copies thereof shall be delivered to any person applying for the same on payment of such sum not exceeding twopence as the Council shall direct and such regulations when so published shall come into operation upon a date to be fixed by the Local Government Board in allowing the regulations and the production of a printed copy of such regulations authenticated by the seal of the Council shall be evidence of the existence and of the due making allowance and publication of such regulations in all

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prosecutions or other proceedings under the same without adducing proof of such seal or of the fact of such making confirmation allowance of publication of such regulations.

**This Part of Act and regulations to form part of Part VI of London Building Act 1894.**

**24.**—The foregoing provisions of this Part of this Act and any regulations in force thereunder shall be deemed to form part of Part VI of the London Building Act 1894 and this Part of this Act and any references in the principal Acts to the said Act of 1894 or any Part thereof shall be construed accordingly.

**Tribunal of appeal, etc.**

**25.**—(1) For the purposes of this Part of this Act the tribunal of appeal shall consist of the three members of the tribunal of appeal from time to time appointed under section 175 of the London Building Act 1894 and of one member appointed by the Council of the Institution of Civil Engineers.

(2) In the event of their being an equality of votes on the tribunal of appeal on any matter arising under this Part of this Act the member acting as the chairman of such tribunal for the time being shall have a second or casting vote.

(3) Regulations made or to be made under section 184 of the London Building Act 1894 shall apply to appeals under this Part of this Act to the tribunal of appeal.

(4) Subject to the provisions of this section the provisions of section 156 and sections 175 to 186 (inclusive) of the London Building Act 1894 shall apply to the tribunal of appeal and appeals thereto under this Part of this Act as if the tribunal of appeal referred to in those sections were the tribunal of appeal constituted by this Part of this Act.

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26.—(1) Where under the provisions of this Part of this Act or any regulations in force thereunder any building is erected or any addition or alteration or other work is made or done to or on any building the district surveyor shall be entitled with regard to such building addition alteration or other work to a fee equal to two and a half times the amount of the fee specified with regard to new buildings in Part I. of the Third Schedule to the London Building Act 1894 calculated as follows:—

Increased fee  
to district  
surveyor in  
certain cases.

*New Buildings.*

Upon the area and height of the building as specified in the said Part I. of the said Third Schedule.

*Additions.*

Upon the area and height of the addition (including therein such portion of the building as may be structurally affected by any alteration or other work necessitated by or involved in the making of the addition) as if such addition had been a new building of the same area and height.

*Alterations and other Works.*

Upon the area and height of the portion of the building structurally affected by the alteration or other work (not necessitated by or involved in the making of an addition) as if such portion had been a new building of the same area and height.

Provided that in calculating the fees payable under this section no regard shall be had to the proviso contained in the said Part I. of the said Third Schedule.

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Increased fee  
to district  
surveyor in  
certain cases.

Provided also that the fees payable to the district surveyor in respect of arches or fire-resisting floors over or under public ways the formation or closing of opening in party walls and on chimneys and flues shall be those specified in the said Part I. of the said Third Schedule.

(2) One-fifth of the amount of any fee payable under this section shall be paid to the district surveyor at the time when notice is served on him under section 145 of the London Building Act 1894.

(3) Subject to the provisions of this section all the provisions of the London Building Act 1894 relating to the payment to and recovery by the district surveyor of fees shall extend and apply to the fees provided for by this section.

Saving existing  
powers and  
rights.

27. The provisions of this Part of this Act and any regulations in force thereunder shall not apply in the case of the erection or alteration of or the making of an addition to or the doing of other work to in or upon any building in accordance with the provisions of the principal Acts and nothing in this Part of this Act or in any regulations in force thereunder shall take away or prejudice any powers rights privileges or exemptions vested in or enjoyed by any person under the principal Acts or any of them.





